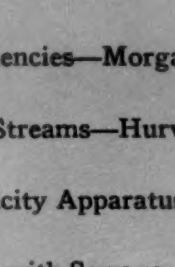
Journal

WATER POLLUTION CONTROL FEDERATION

FORMERLY SEWAGE AND INDUSTRIAL WASTES



Special Features

Diffuser Efficiencies—Morgan and Bewtra

ABS in Streams—Hurwitz et al.

Fish Toxicity Apparatus—Grenier

Canning Waste with Sewage—Norgaard et al.

Activated Sludge in Germany—Kehr and von der Emde

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Operator's Manual

OCTOBER 1960

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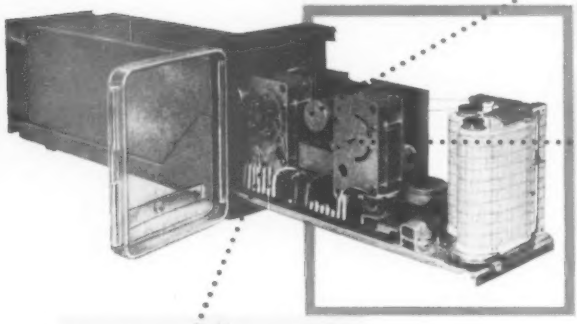

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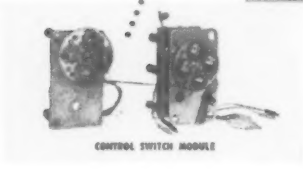

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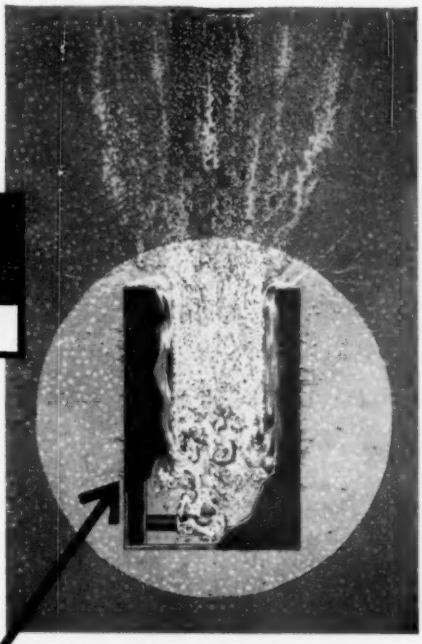

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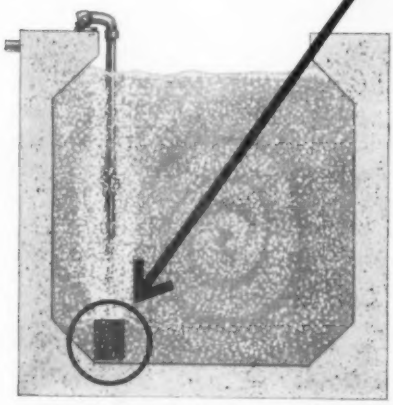

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Volume 32 October 1960 Number 10

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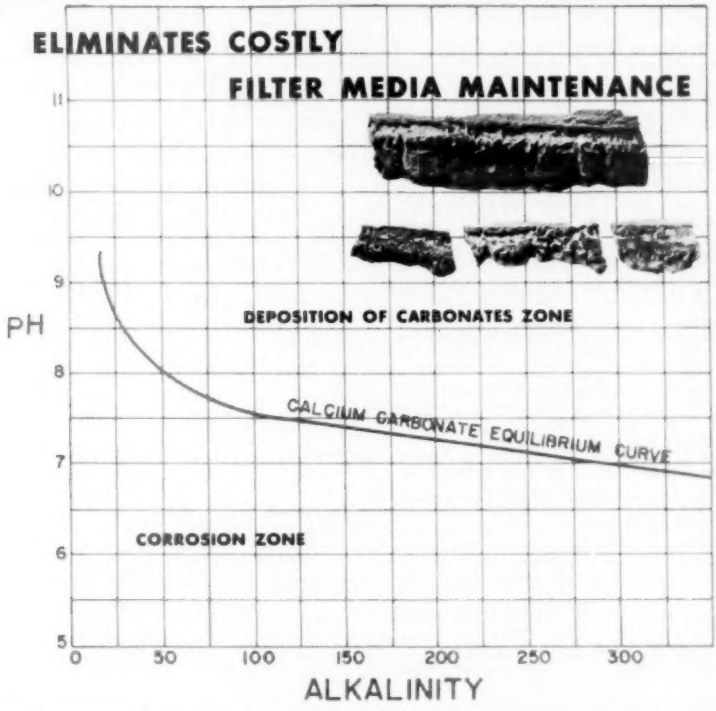

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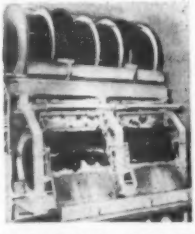

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Indiana	Nov. 16–17	Hotel Washington, Indianapolis
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Mississippi	March 10, 1961	Robert E. Lee Hotel, Jackson

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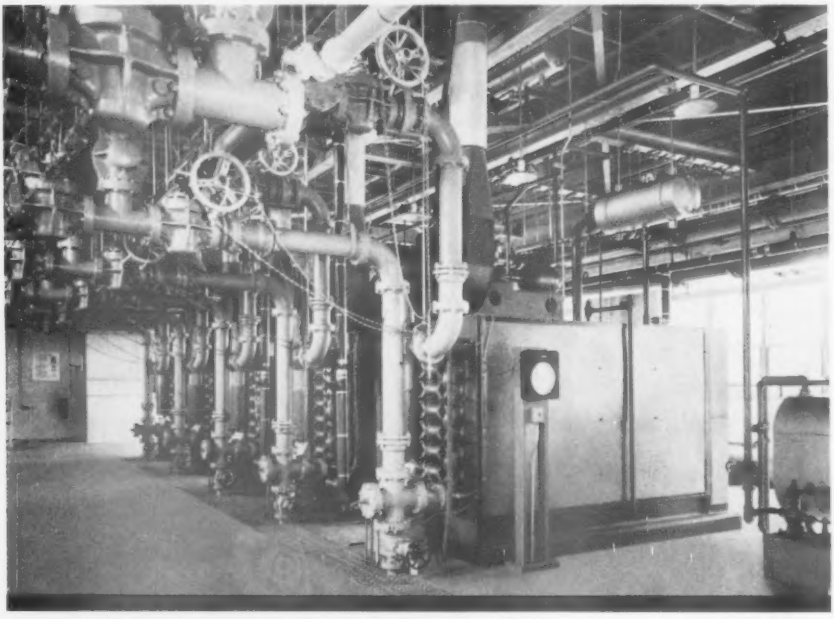
View of Sludge Heating and Digestion Building. Metal was shop-primed with Inertol Rustinhibitive Primer 621. Masonry wall and ceiling areas were primed with Ramuc® Utility Undercoater. Ramuc Utility Enamel was then applied to all primed surfaces. Special Inertol high-heat paints were used where required. Here—and in many other areas throughout the entire plant—Inertol specialized coatings will protect efficiently, attractively, economically...for *many* years.



CINCINNATI'S GIANT SEWAGE PLANT PROTECTED BY INERTOL® PAINTS

Because this plant is centered in a highly industrialized area, it must cope with liquid wastes about twice as strong as the average municipality: suspended solids, 500 ppm; 5-day BOD, 400 ppm. This calls for maximum plant protection—one of the important reasons Consulting Engineers Havens & Emerson specified Inertol paints.

Mr. Arthur D. Caster, Cincinnati's Principal Sewage Disposal Engineer, comments: "Our experience has proven that only the best protective coating should be used initially in Sewage Treatment Works, in order to reduce to a minimum maintenance costs during the future years." The majority of the painted surfaces in this new plant, serving Cincinnati and twenty-six surrounding communities, are coated with Inertol products. Each paint is designed specifically for a particular surface.

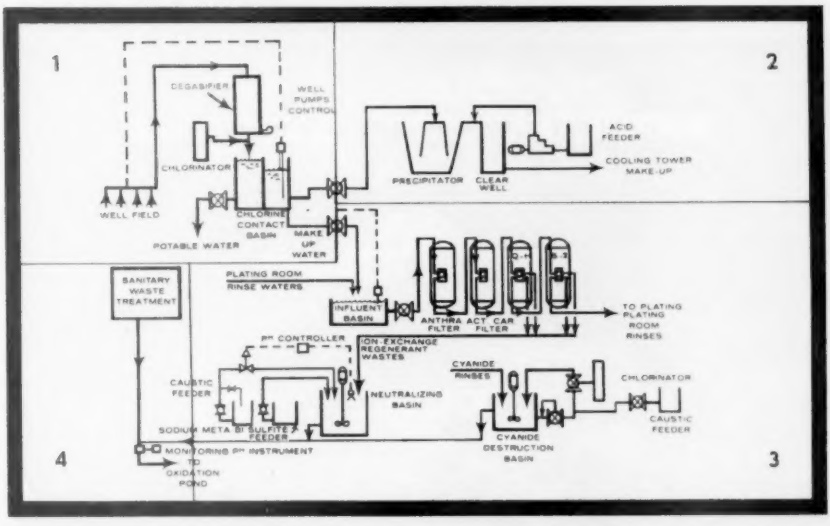
Inertol Company would welcome the opportunity to study *your* requirements. Write for complete paint specifications covering your current project. There's no cost or obligation, of course. Also—be sure to ask for free, valuable maintenance painting guide SI 586.

A Complete Line of Quality Coatings for Sewage, Industrial Wastes, and Water Plants and Swimming Pools.

Inertol® **INERTOL CO., INC.**

482 Frelinghuysen Avenue, Newark 12, New Jersey • 27H South Park, San Francisco 7, California

FLUIDICS* AT WORK



How many of these water and waste treatment ideas can you use profitably?

From well point to waste disposal this new system now serving the Electro-Mechanical Research Company, Sarasota, Florida, is packed with ideas that can help *you* solve similar problems—and each idea was brought to reality with equipment supplied by Permutit, as prime contractor:

1. Well-water treatment (upper left in diagram) handles a minimum flow of 160 gallons per minute from a well field—84 gallons for industrial cooling, 10 gallons for plating rinse make-up, and the balance for potable use. All this raw water is degasified and treated in a Permutit equipment system.

2. Cooling water undergoes further treatment (top right) for reduction of carbonate and bicarbonate alkalinity, by means of a Permutit® Precipitator and acid feeder.

3. Plating room rinse-water demineralizing (center and lower right). A complete Permutit system saves water, saves heat, treats regenerant wastes, and provides high-quality demineralized water for plating rinse. Make-up and plating rinse water is processed through an anthracite filter to remove suspended solids, a Carbo-Dur filter to remove organics, a Permutit Q-H cation exchanger, and a Permutit S-2 anion exchanger. This demineralized water is then ready for use. After plating rinse operations, water is recycled, treated and re-used. Wastes —including cyanide rinses, regenerants, and others—are batch treated.

4. Sanitary waste treatment (lower left) was designed to handle 20,000 gallons per day, treating them first in an Imhoff tank, followed by high-rate bio-filtration, secondary clarification and chlorination.

The entire system was designed by Smally, Wellford and Nalven, Sarasota, Florida, consulting engineers. *Your* engineering consultants can help you obtain similar results. Talk it over with them, or write for our Bulletin 4433, "Water Treatment Equipment," and/or Bulletin 4486, "Equipment for Treating Industrial Wastes."

Permutit Division, Dept. JW-100, 50 West 44th Street, New York 36, New York.

PFAUDLER PERMUTIT INC.
Specialists in FLUIDICS... the science of fluid processes

*FLUIDICS is the Pfaudler Permutit program that integrates knowledge, equipment and experience in solving problems involving fluids.

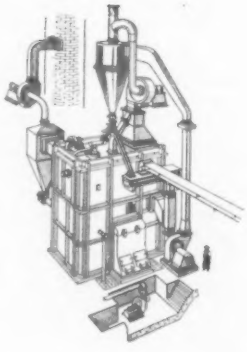
Dispose of Sewage Sludge the Modern Way

The widespread acceptance of the C-E Raymond System is based on its solid advantages. This compact system combines the principles of Flash Drying and Incineration in a unit in which filter cake can be reduced to a sterile ash . . . or flash-dried to a salable fertilizer and soil conditioner. Revenue from the latter is often sufficient to pay a substantial part of plant operating costs.

Either operation can be performed alone, or combined in any desired proportion. Effective high-temperature deodorization equipment to remove foul odors from combustion gases is available for both processes.

A C-E Raymond System can mean an end to your community's sludge disposal problems . . . as it has in municipalities across the nation. Contact the Combustion office nearest you for specific recommendations. A C-E specialist will be glad to discuss your requirements and help plan an installation with you or your consultants.

B-998C



COMBUSTION ENGINEERING

RAYMOND DIVISION, 427 West Randolph St., Chicago 6, Illinois
Eastern Office: 200 Madison Avenue, New York 16, N. Y.
Western Office: 510 West Sixth Street, Los Angeles 14, Cal.
Canada: Combustion Engineering-Superheater Ltd.

ALSO FLASH DRYING AND INCINERATION SYSTEMS FOR INDUSTRIAL WASTE DISPOSAL

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90% of the Cities in the United States use this ROD for cleaning sewers . . .

THERE MUST BE A REASON!

- BEST BY LABORATORY AND FIELD TEST
- SPECIAL STEEL FOR SEWER RODDING
- ACID RESISTANCE TREATED
- GROOVED COUPLINGS

FLEXICROME RODS are the end result of many years of field and laboratory tests. When you specify "**Flexicrome**" you get these important, exclusive features:

* **Special alloy steel for extra toughness and durability.**
* **Heat fused rod coating for better sewer acid resistance.**
* **Solid center, grooved couplings for extra rod end support, and faster assembly.**

Patent No. 2,471,060

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LIMA, OHIO DALLAS, TEXAS

PIPE THAT DOESN'T GET COLD FEET IN WET TRENCHES

This cast iron pipe was installed to carry water to an important industrial site in Indiana.

At the time of installation, weather conditions were severe; the trenches were wet and muddy.

Still, the cast iron pipe was easily and speedily assembled, enabling the contractor to complete the job well ahead of the deadline.

Cast iron pipe does not get cold feet over rugged soil conditions. Its rugged design, corrosion-resistant qualities, and bottle-tight joints will withstand the most severe conditions.

Modern, cement-lined cast iron pipe will deliver a continued full flow of water for at least a century . . . and rarely requires repairs or replacement.

CAST IRON PIPE RESEARCH ASSOCIATION
Thos. F. Wolfe, Managing Director
3440 Prudential Plaza, Chicago 1, Illinois

CAST IRON PIPE
THE MARK OF THE 100-YEAR PIPE

"He musta misunderstood Gran'maw. She told him Tyton is easy to handle!"

U.S. cast iron PIPE FOR WATER, SEWERAGE AND

losing no time!

Not Cousin Willie. He's used to working with Tyton®, fastest, simplest, easiest to install pipe that ever snuggled into a trench.

No bell holes, caulking equipment, nuts, bolts or other bothersome equipment. Tyton Joint® pipe needs only one accessory. No weather worries, either. Tyton can be laid in rain or wet trench if need be.

Lose no time. Get the time and money-saving facts on Tyton Joint pipe. Call or write.

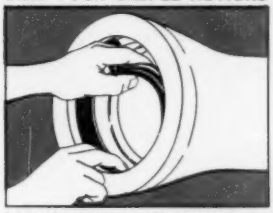
U.S. PIPE AND FOUNDRY COMPANY
General Office: Birmingham 2, Alabama

A Wholly Integrated Producer from Mines and Blast Furnaces to Finished Pipe.

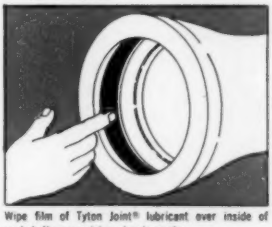
INDUSTRIAL SERVICE CAST IRON

TYTON®

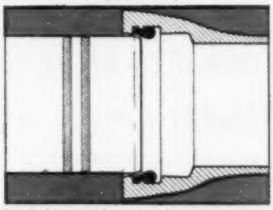
ONLY FOUR SIMPLE ACTIONS



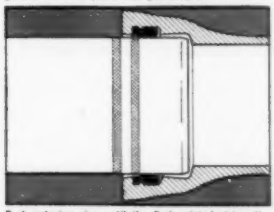
Insert gasket with groove over bead in gasket seat . . . a simple hand operation.



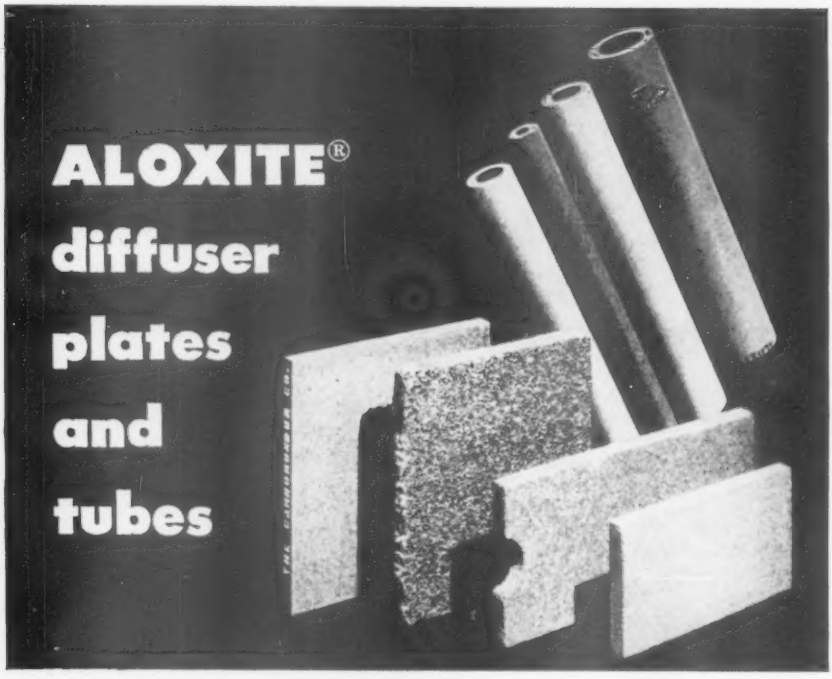
Wipe film of Tyton Joint® lubricant over inside of gasket. Your receiving pipe is ready.



Insert plain end of entering pipe until it touches gasket. Note two painted stripes on end.



Push entering pipe until the first painted stripe disappears and the second stripe is approximately flush with bell face. The joint is sealed . . . bottle-tight, permanently! The job's done . . . . fast, efficiently, economically. Could anything be simpler?



Your assurance of uniform dispersion . . . long diffuser life . . . trouble free service

ALOXITE aluminum oxide diffusers have proved successful in hundreds of installations – some with more than 25 years of service. ALOXITE installations outnumber all other makes of porous ceramic diffusers *combined*.

Remember these characteristics of ALOXITE when considering your air diffuser requirements . . . (1) A uniform grain and pore structure. This assures uniform, small-bubble diffusion. (2) A smooth, low-resistance surface. The glass-like ceramic bond that coats each grain minimizes pressure loss or clogging. (3) Permanent resistance to corrosion. (4) Adaptability to practically all cleaning methods. (5) Great strength and ruggedness.

ALOXITE diffusers are available in plate or tube form and in a full range of grades. This permits maximum flexibility of design . . . your assurance of aeration results.

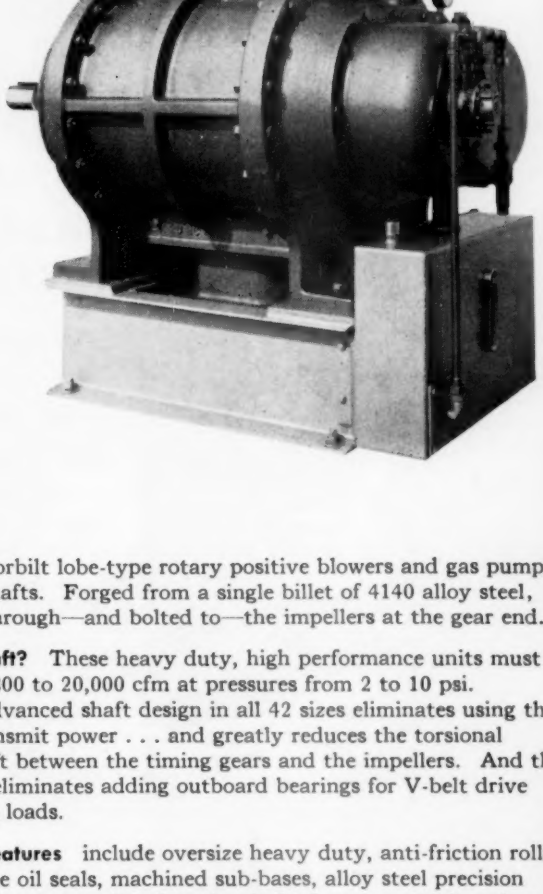
Write for free 56-page booklet to Dept. N-100, Refractories Division, The Carborundum Co., Perth Amboy, N. J.

For time-tested porous media . . . count on

CARBORUNDUM®

NEW SUTORBILT SERIES 3200 BLOWERS AND GAS PUMPS

with one-piece shafts
for greater efficiency at increased
horsepower and pressure ratings



New Series 3200 Sutorbilt lobe-type rotary positive blowers and gas pumps feature *one-piece* shafts. Forged from a single billet of 4140 alloy steel, they are slip fitted through—and bolted to—the impellers at the gear end.

Why a One-Piece Shaft? These heavy duty, high performance units must operate safely from 800 to 20,000 cfm at pressures from 2 to 10 psi. Incorporating this advanced shaft design in all 42 sizes eliminates using the drive impeller to transmit power . . . and greatly reduces the torsional deflection of the shaft between the timing gears and the impellers. And this one-piece shaft also eliminates adding outboard bearings for V-belt drive with normal working loads.

Other Outstanding Features include oversize heavy duty, anti-friction roller bearings with lip-type oil seals, machined sub-bases, alloy steel precision machined helical timing gears piloted to the shafts for concentricity, close grain cast iron impellers and a reversible oil pump with an oversize sump that eliminates the need for an oil cooler at ambient temperatures below 125 °F. In gas and high vacuum pumps, mechanical seals prevent leakage or gas contamination.

For more information and detailed specifications on these powerful, peak-performance units, write today, Department E.

Representatives in principal cities. Consult your classified telephone directory

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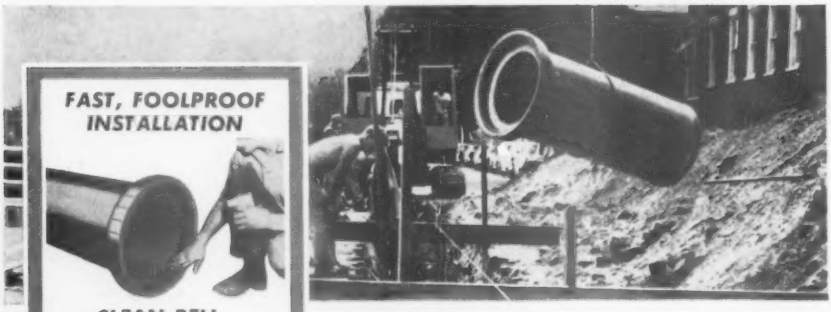

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PLACE RUBBER "O" RING IN SPIGOT GROOVE

LINE UP, SHOVE INTO PLACE THAT'S ALL THERE IS TO IT!

TODAY'S

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Wedge-Lock* Type "O"

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Now, lines go in faster, seal tighter, stay trouble-free longer. Robinson Type "O" joints reduce laying time . . . cut installation costs . . . are ideally suited to difficult installation conditions. New precision-tight Factory-Made Wedge-Lock Type "O" joints handle infiltration and roots in any terrain.

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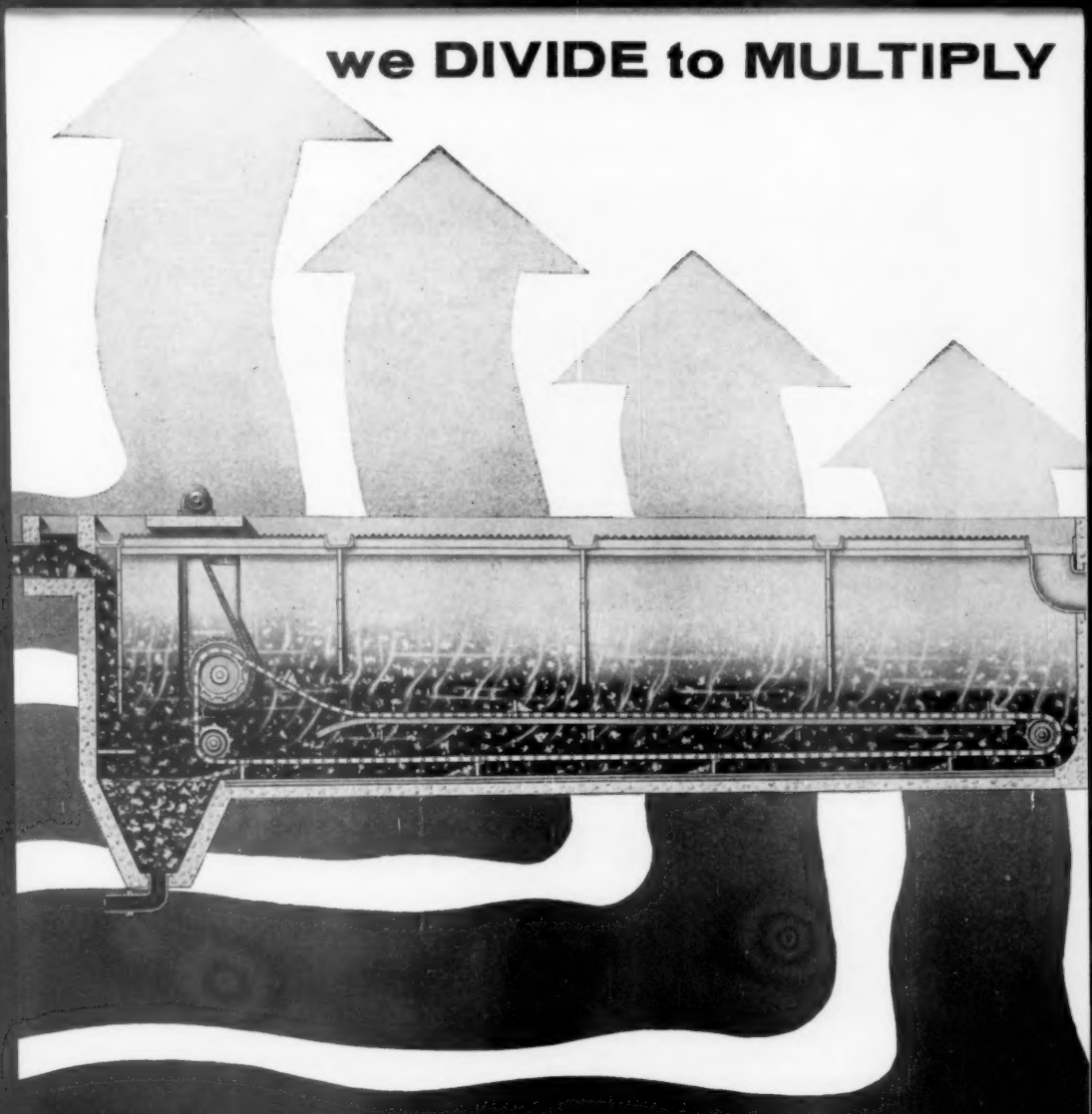
we DIVIDE to MULTIPLY



we DIVIDE the flow to MULTIPLY clarifier capacity

Rex Verti-Flo not only increases the clarifying capacity of tanks up to four times; it provides a far clearer effluent. The savings in equipment and construction costs are obvious. Here's how Verti-Flo does it: The unique design of Rex Verti-Flo Clarifier transforms the conventional horizontal-flow tank into a vertical-flow tank...utilizing the *full volume* of the tank and *minimizing short-circuiting*. This is accomplished by a system of collecting troughs, weirs and partitioning baffles which divide the large, horizontal settling zone into a series of small, vertical-flow cells. To assure maximum flow length and control, the weirs are adjustable entirely around the periphery of each cell.

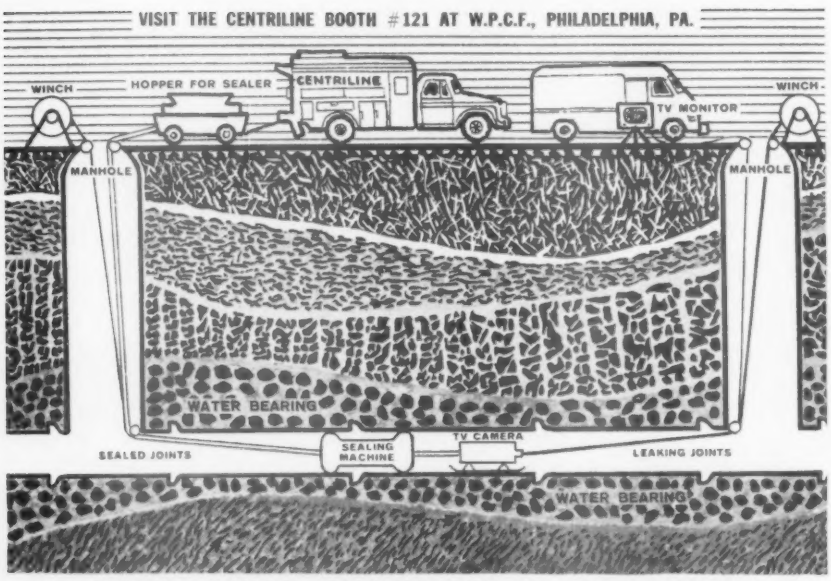
Installed in existing tanks, Verti-Flo can be at least double the capacity of the tank...in new installations, Verti-Flo provides up to four times more capacity so that smaller basins can be used with considerable savings in both equipment and construction costs.

For information, write CHAIN Belt Company, 4606 W. Greenfield Ave., Milwaukee 1, Wis.

REX®

CHAIN BELT COMPANY

VISIT THE CENTRILINE BOOTH #121 AT W.P.C.F., PHILADELPHIA, PA.



NEW! THE CENTRISEAL PROCESS ELIMINATES LEAKING JOINTS

Now joint leakage can effectively be stopped in small as well as large sanitary and storm drains. By remote control—constantly monitored by television which finds the leaks and checks the seal afterwards—special CENTRISEAL PROCESS equipment forces a fast-setting polymeric sealer into open joints **from the inside.** This forms a tough seal in the pipe joint as well as outside in the surrounding soil. You're thus assured a permanent bond, and leakage from either internal or external pressures is prevented. No excavations are necessary and the grouting process is remotely controlled from above ground. Savings are found in maintenance and pumping costs and capital expenditures are avoided.

INCIDENTALLY... If your problem is more than leaking joints, if you have deteriorated pipes, investigate the other Centriline services. The CENTRILINE PROCESS lines pipes with a cement mortar or acid-resistant epoxy mortar applied centrifugally. Because the pipes remain in place, there's no traffic interruption. Pipe life is increased, ground and pavement settlement due to infiltration eliminated. Write now for complete information on the CENTRISEAL PROCESS for leaking joints and the CENTRILINE PROCESS for lining pipe.

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Subsidiary of Raymond International Inc.

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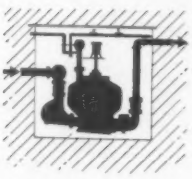
Branch Offices in Principal Cities of the United States, Canada and Latin America

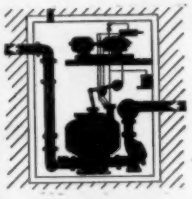
Which sewage lift station is best for your job?

TOPS IN DEPENDABILITY AND SAFETY



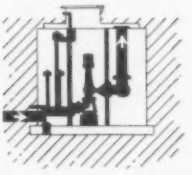
Concrete or steel underground station with Shone® Type S pneumatic ejector, mechanically controlled (warranted 25 years). Remote air supply. Nothing electrical; maintenance free; operates even under water. For municipalities and large subdivisions.

SUPRISINGLY VERSATILE, INEXPENSIVE



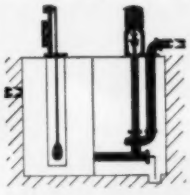
Shone® Type SAC station, underground in concrete vault. Mechanically controlled ejector (warranted 25 years) has its own compressor. Frequently less costly than package stations. Finds use everywhere.

PACKAGE STATION



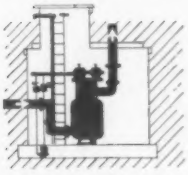
Quality underground station with vertical, short-coupled (Series 6260), dry pit centrifugal pumps. Quality design and equipment give good service life at nominal cost. For small permanent stations, growing areas.

LOW FIRST COST—EASY INSTALLATION



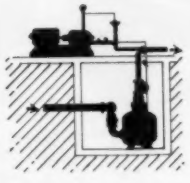
Two-compartment steel basin with centrifugal vertical wet pit pump. Pumps are not submerged; motors protected from flooding. Low first cost; fast installation. For buildings, small growing subdivisions.

ULTIMATE IN PACKAGE DESIGN



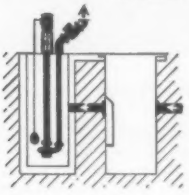
Package underground station with Expelsor® pneumatic ejector, electrode controlled. Sanitary and dependable, low maintenance, easy to service. Recommended for municipal and subdivision applications.

BUILDING TYPE STATION



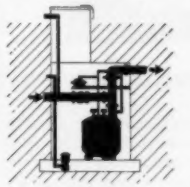
Expelsor® pneumatic ejector station designed for use in buildings. Compressor is on basement floor; ejector on floor below. Sealed installation gives safe, odorless handling of sewage indoors.

SPECIAL PROBLEMS?



Centrifugal vertical wet pit pumps with adjoining settling basin. Good for areas where sand or old sewers create problems. Special pump construction for abrasive or corrosive conditions, industrial wastes.

MAXIMUM RELIABILITY—MINIMUM SPACE



Package underground station. Expelsor® pneumatic ejector has two compartments, duplex compressors and controls. Excellent service accessibility. Especially suited for subdivision and municipal lift stations.

The answer depends on the job. Is it to be a temporary station? What about future loads? How much maintenance will it get? What type of power is available? These and many other questions must be weighed carefully and without bias. In making the decision, why not put Yeomans 61 years' experience (59 on underground stations alone) to work? Because Yeomans builds *the only complete line* of sewage lift stations in the industry, recommendations are *impartial* and in your best interest. Phone, wire, or write now.

L-5911

YEOMANS

1999-7 N. Ruby St., Melrose Park, Ill.

SPECIALISTS IN SEWAGE LIFT STATIONS—THE COMPLETE LINE

PARSHALL FLUME LINERS

PREFABRICATED TO SPEED CONSTRUCTION

● Ready for field installation! Parshall Flume Liners are prefabricated from "Chemstone E," an asbestos cement material impregnated with a special asphaltic compound under tremendous pressure. It will withstand shock, vibration and is impervious to most acids, alkaline solutions and rapid temperature changes. More durable than a concrete channel. Although strong enough to stand alone, it is generally used as a liner in concrete channels and serves as the inside form during the pour.

Tolerances are held within plus or minus $\frac{1}{16}$"—well within the limits of accuracy for this type of metering.

Flume liners with throats 3" to 12" are furnished on skids completely assembled for quick, easy installation. For throats 18" to 96" the liners are furnished in two sections.

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275 HOLLENBECK STREET, ROCHESTER 21, N. Y.

Specify IOWA for every job

Iowa's complete line of valves for treatment plants assures you of efficient operation, low maintenance cost, and replacement parts when needed in years to come.

IOWA CHECK VALVES—Swing gate type for use in either horizontal or vertical pipe lines. Bronze trimmed throughout. Stainless steel hinge pins. Leather or rubber faced gates, or solid bronze, as desired—aluminum gates available for air service.

IOWA MEDIUM AND LOW PRESSURE VALVES—either non-rising stem or outside screw and yoke types. Designed as carefully as those conforming to A.W.W.A. specifications but with lighter construction for lower pressures.

Write TODAY for descriptive literature

IOWA FLAP VALVES—all iron; can be furnished with bronze hinge bolts or fully bronze mounted—including bronze bolts, flap ring and seat ring—when required.

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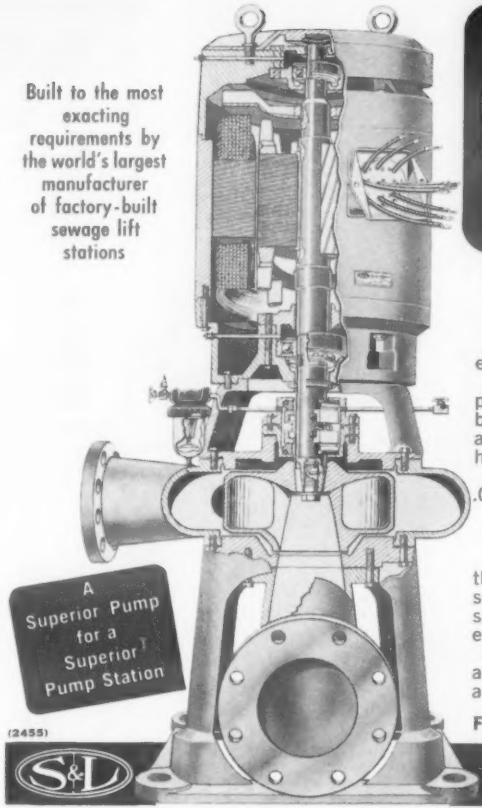

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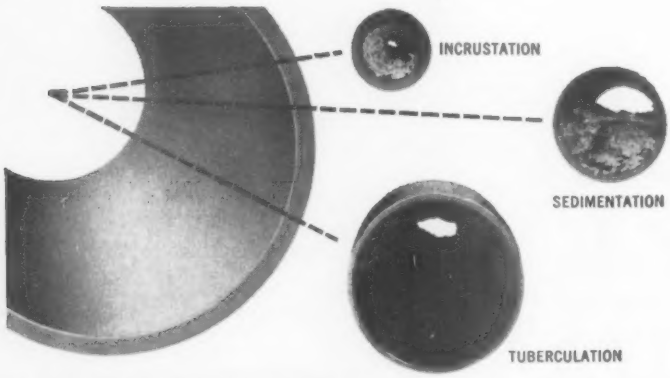

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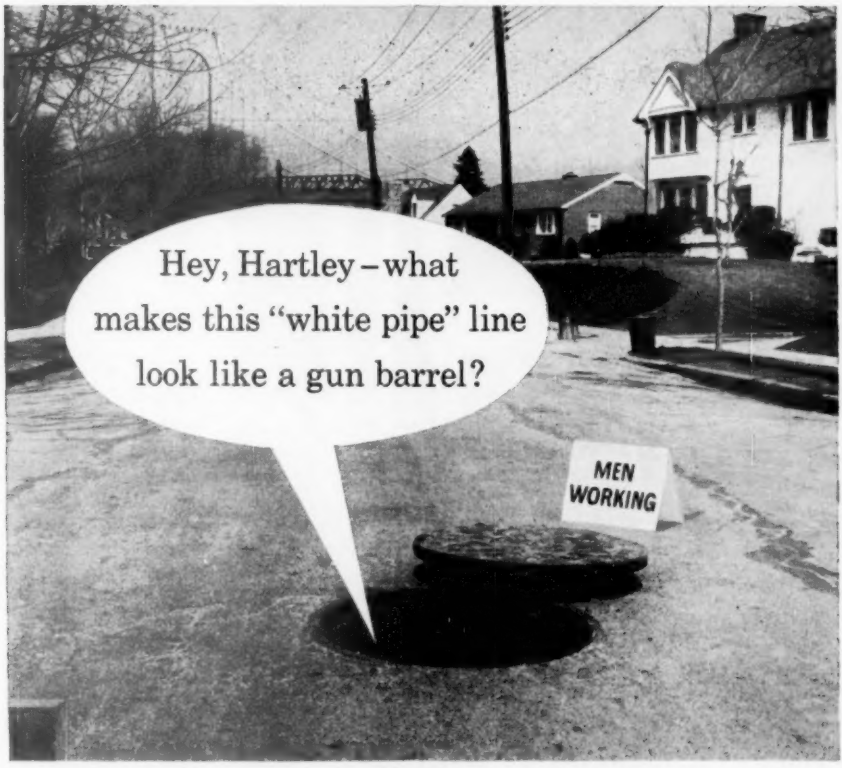

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AIR DIFFUSER EFFICIENCIES *

By Philip F. Morgan and Jatinder K. Bewtra

Respectively, Professor of Sanitary Engineering, and Research Assistant, State University of Iowa, Iowa City, Iowa

Air diffusion is an important unit operation which has been used in sewage and industrial waste treatment for many years but it is only recently that test procedures have been developed to evaluate accurately the operating efficiencies of various diffusion systems. Even at the present time there has not been any general acceptance of standard test procedures. In some cases it appears that diffusers have been rated for their oxygen absorption efficiencies on the basis of oxygen absorbed by a specific biological suspension. Such a result is actually a process efficiency and measures both the absorption capacity of the biological suspension and the diffuser efficiency. Such a rating can be compared to testing the pumping efficiency of a centrifugal pump while pumping a 5- or 10-per cent slurry and expecting it to agree with the efficiency of the same pump handling clear water and under standard test conditions. Such data may be extremely important but it is not "standard" data which can be used to compare either the diffuser or the pump under other conditions of application.

Oxygen absorption efficiencies of diffusers are of particular interest for the newer accelerated biological treatment processes because of the much greater oxygen demand rates and in the more conventional plants because, with the exception of labor, the cost of compressing the air is often the largest single operating expense. This study was undertaken to: (*a*) evaluate the absorption efficiencies of various commonly used air diffusion devices under standard conditions that would be applicable to full scale installations, (*b*) obtain basic information about the diffusion process which could be used to improve aeration efficiencies, and (*c*) present the results in a form that would be useful to the designing engineer.

The only unit of measure applicable to the evaluation of all types of aeration devices is the quantity of power required to dissolve a unit weight of oxygen. This power efficiency must be used for air diffusers with varying submergences and for all types of mechanical aeration devices. However, since most aeration installations are of the diffused air type with a fixed submergence, it is possible to simplify the evaluation by comparing the weight of oxygen absorbed by the liquid to the weight of oxygen supplied in the air to the diffuser. This eliminates the need for considering the efficiency of the air compressor, a variable independent of the diffuser system.

For the purpose of this study the term "diffuser efficiency" is used to indicate the quantity of oxygen absorbed by the liquid in per cent of

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

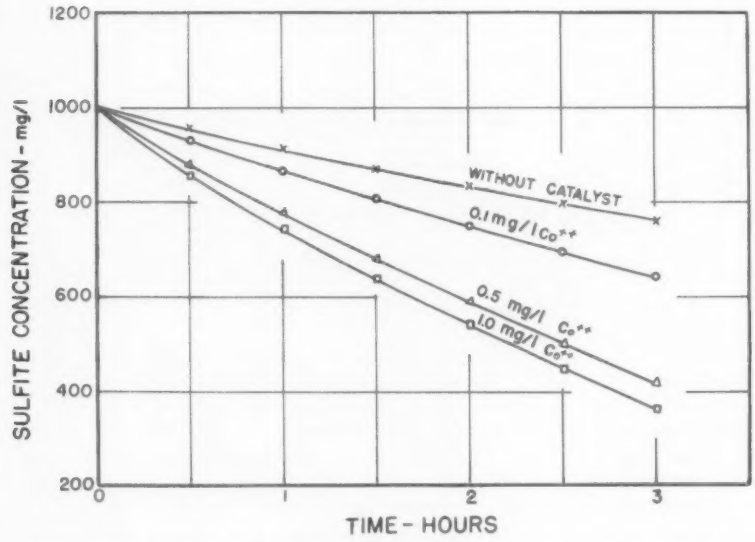

FIGURE 1.—Effect of catalyst concentration on rate of sulfite depletion in laboratory test tank.

that supplied to the diffuser at "standard" conditions.

Selection of Standard Conditions

Important variables involved in establishing "standard" conditions for testing diffusers efficiencies include: (*a*) absorbing liquid; (*b*) method of measuring, computing, and reporting rate of oxygen uptake; (*c*) temperature of absorbing liquid; and (*d*) geometry of test tank.

The Absorbing Liquid

The first step in establishing standard test conditions is to select the liquid into which the oxygen is to be absorbed. One method is to diffuse air into a solution of sodium sulfite and measure the rate of depletion of the sulfite. This is chemically simple and easily understood and has been widely used in the past. A copper or cobalt catalyst generally is used to increase the absorption rate of the sulfite. Experience with sulfite has, however, indicated that it is difficult to duplicate results. Figure 1 shows the effect of varying the concentration of cobalt chloride as a catalyst in a small laboratory diffused air test tank. All conditions, including rate of air flow, temperature, sulfite concentration, and diffuser were the same for all runs. It is obvious that the rate of absorption is a function of catalyst concentration. Temperature and sulfite concentrations also affect absorption rates. It is apparent from this and other tests that sodium sulfite simply cannot be utilized for "standard" test conditions. The sodium sulfite method is, however, a convenient method for creating non-standard conditions in the laboratory to simulate certain biological suspensions having an exceedingly high oxygen demand.

The direct measurement of oxygen utilization in a biological suspension is likewise unsatisfactory for comparing diffusers in different locations because this is actually a process absorption. Similar to sulfite with varying catalyst concentrations, the rate of oxygen uptake varies from time to time depend-

ing on the quality or demand of the suspension, and thus a biological suspension cannot be considered "standard."

Tap water has been used as the oxygen absorption media. It has proved very satisfactory in the current study and is recommended as a "standard" media. Additional work is indicated to determine the exact variation in absorption rate for varying water quality but it appears that this variation will be well within the limits of acceptable accuracy.

Measuring, Computing, and Reporting the Rate of Uptake

The two-film theory explaining the phenomenon of transfer of oxygen from the bubble to the liquid was explained by Louis and Whitman (1) 25 years ago but it was not until 1952 that Ippen *et al.* (2) presented their detailed study showing the application of this theory for the computation of oxygen absorption rates in water. This method has been further described and developed by Gaden (3), Zieminski *et al.* (4), and others.

When the two-film theory is applied for measuring oxygen absorption rates a quantity of water is first deoxygenated until the dissolved oxygen is reduced to approximately zero. Aeration is then started and the rate of oxygen uptake is measured. For this study the water was deoxygenated by adding a slight excess of sodium sulfite in solution with cobalt catalyst. Bewtra (5) has shown that with the cobalt catalyst, oxidation is so rapid that the dissolved oxygen remains at zero until the sulfite is completely exhausted. Thus, there is no appreciable error resulting from the sulfite being present just before the oxygen absorption curve is started. However, the number of runs completed with one tank of water is limited to avoid excessive accumulation of sodium sulfate in the water.

After aeration is started the rate of increase of dissolved oxygen is measured by taking a series of dissolved oxygen samples at regular intervals and titrating for dissolved oxygen according to the azide modification of the Winkler method as outlined in "Standard Methods" (6). This method is satisfactory for full scale work where the volume of sample used is insignificant in relation to the total media

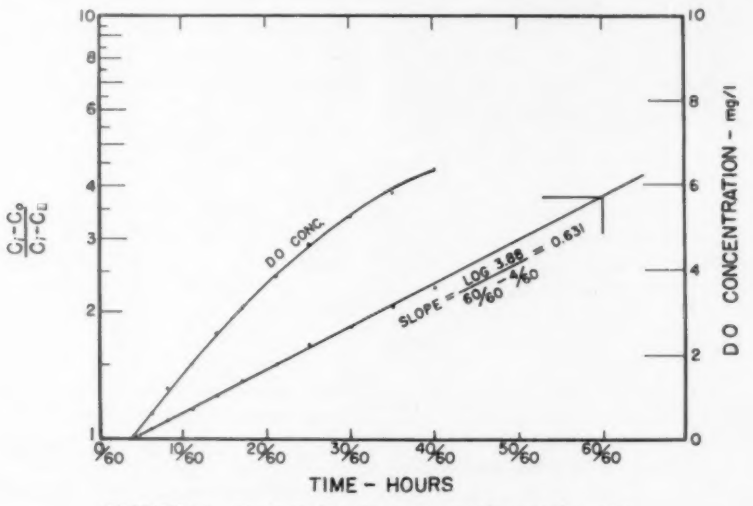

FIGURE 2.—Typical plot of oxygen uptake rate in water.

volume. For smaller laboratory aeration tanks, the polarographic method as outlined in "Standard Methods" is preferable.

Computations for the rate of oxygen absorption are based on the basic relation presented by Ippen *et al.* (2).

$$\frac{dc}{dt} = \frac{10^6}{W} KA\,(C_i - C_L) \quad \ldots (1)$$

where

$\frac{dc}{dt}$ = Rate of change of dissolved oxygen concentration in the liquid mg/l/hr;
W = weight of water in tank, lb;
A = interfacial or absorbing area, sq ft;
K = coefficient of oxygen absorption, lb oxygen/hr × sq ft × unit concentration difference in mg/l;
C_i = equilibrium concentration of oxygen at the phase interface, mg/l (assumed to be saturation at mid-depth over diffuser); and
C_L = concentration of oxygen in the liquid at time t, mg/l.

Integrating this relation gives:

$$\log_{10} \frac{C_i - C_o}{C_i - C_L} = \frac{10^6}{2.30W} KAt \quad (2)$$

where

C_o = initial concentration of oxygen in liquid, mg/l. (For plotting purposes the axis is adjusted so that $C_o = 0$ at $t = 0$. Thus $(C_i - C_o) = C_i$ = saturation at mid-depth over diffusers.); and
$(C_i - C_L)$ = oxygen saturation deficit in the liquid, mg/l.

By plotting $\frac{C_i - C_o}{C_i - C_L}$ vs. time on semilog paper the value $\frac{10^6}{2.30W} KA$ is determined from the slope of the plotted values. Figure 2 shows a typical plot. Then $\frac{dc}{dt}$ = slope $(C_i - C_L)$ 2.30.

If C_L is taken as zero, the rate $\frac{dc}{dt}$ is the rate of oxygen absorption into the water in mg/l/hr at zero dissolved oxygen. The diffuser efficiency is then computed from the rate of oxygen contained in the air input to the tank and the rate of oxygen absorbed into the water (at zero dissolved oxygen).

Temperature of Absorbing Liquid

Twenty degrees centigrade was apparently adopted by Ippen *et al.* (2) and Zieminski *et al.* (4) for their work and since this is also the temperature for standard BOD work, it has been adopted as the standard temperature for this study.

Geometry of Test Tank

The importance of the size and shape or geometry of the test tank on the oxygen absorption efficiencies has been indicated by Eckenfelder (7), so that it was apparent that this factor should be considered in any evaluation of diffusion devices. A study of the effect of geometry was undertaken in the special two-column aeration tank (8) shown in Figure 3. Each column is 9 in. square and when in normal operation the water depth is maintained 7 in. deep in the head tank to give a total water depth of 11 ft 2 in. The diffuser under study is mounted in the left column, 1 ft 9 in. above the tank bottom to provide a submergence of 9 ft 5 in.

In operation, the air-water mixture rises in the left column, the air is released in the head tank, and the water returns down the right column. Thus, the two columns of water represent 9-in. square elements in a conventional spiral flow aeration tank, the left column representing an element adjacent to the aeration side wall, and the right column representing an element adjacent to the opposite side wall. Air flow rates were measured with a standard orifice meter and water circulation

rates were measured with a traverse pitometer near the bottom of the right column, as shown in Figure 3.

To operate as a single-column tank, a membrane was placed across the column joint just below the diffuser and the water surface was lowered to below the bottom of the head tank. Figure 4 shows the diffuser efficiency of a porous media diffuser 6 in. long in this tank: (*a*) with water recirculation, two columns operating; (*b*) without recirculation, membrane in place; and (*c*) without recirculation, with the data corrected to the same submergence depth as for the two-column operation. These data indicate that the oxygen absorption with the conventional spiral flow system is only about 60 per cent of the absorption obtained with the same air flow in a single confined column. The reason is that in the confined column there is violent turbulence developed between the centrally rising air-water mixture and the adjacent water flowing down the column to replace that which is carried up in the air-water stream. Bubbles are retained in the downward stream, also larger bubbles are broken in the turbulence and undoubtedly the coefficient K is increased. Although the single column efficiency cannot be duplicated readily in a large areation tank, this comparison does indicate the tremendous effect of tank geometry. It is thus apparent that data from studies conducted with diffusers in small confined tanks cannot be expected to be comparable with data for the same diffuser in a large tank where circulation is free.

Even though it was thought that results from the two-column tank were equivalent to those in a large tank, it could not be considered standard unless it could be compared to results in a full scale tank.

Full Scale Test Tank

The full scale oxygenation test tank was designed to test the oxygen absorption efficiencies of various diffuser systems under "standard" conditions that would be comparable to full scale installations. The use of this tank should eliminate much of the confusion which existed from trying to extrapolate from various scale studies and field tests where "standard" conditions cannot be established.

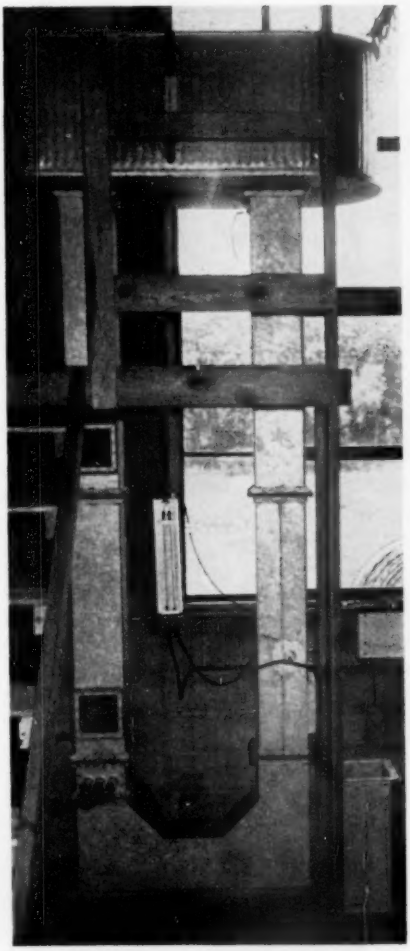

FIGURE 3.—Two-column laboratory aeration tank.

The test tank is a 4-ft long section of a conventional aeration tank, 24 ft wide and 15.5 ft deep, with a capacity

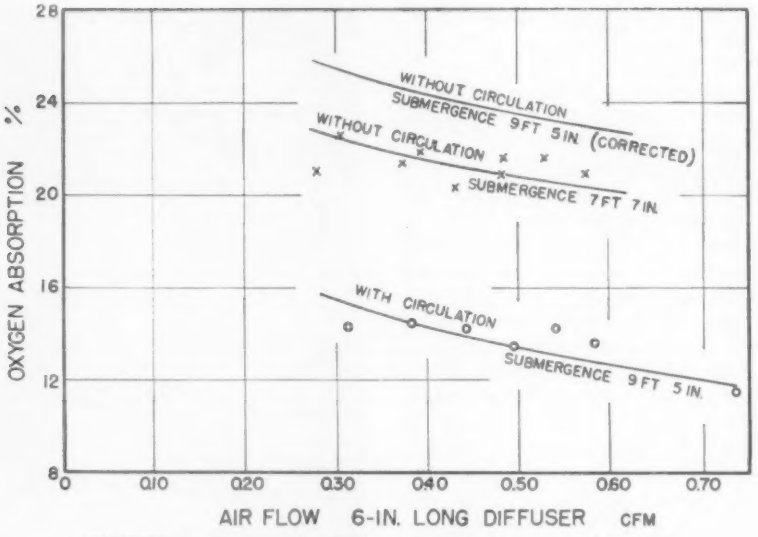

FIGURE 4.—Saran tube efficiency in two-column aeration tank.

of approximately 10,000 gal as shown in Figure 5. Ten windows on the front side and four on the end provide visual inspection of the aeration in progress. City water is used to fill the tank and a circulation system is provided to circulate the water through either of two heat exchangers. One heat exchanger is connected to the laboratory heating system to raise the temperature of the water and the other is connected to a four-ton refrigeration unit to cool it. The air supply for the tank consists of three 40-cfm and one 12-cfm positive displacement blowers connected to a common mani-

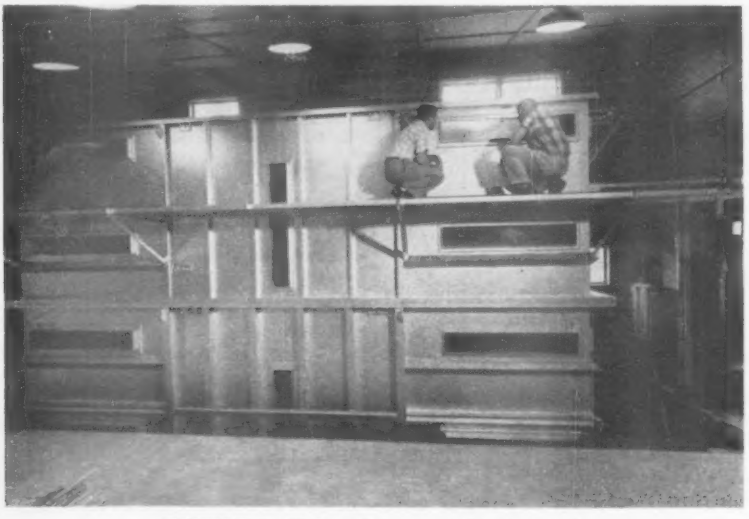

FIGURE 5.—Full scale oxygenation test tank.

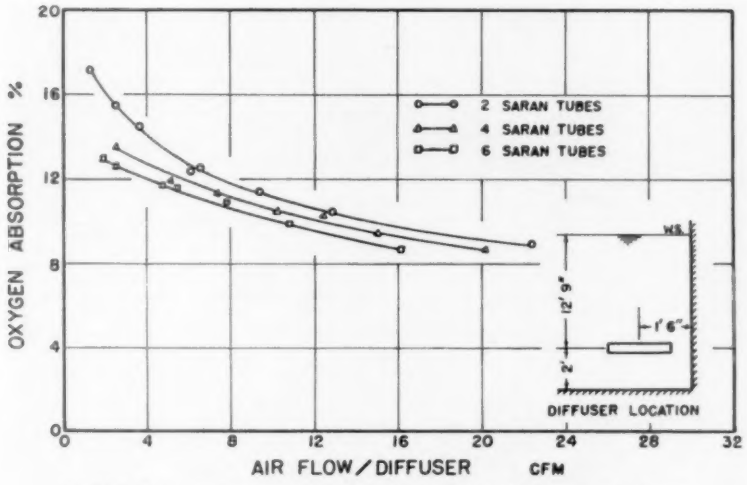

FIGURE 6.—Saran tube efficiency in terms of air flow per diffuser.

fold and metered through an orifice meter. In addition, a 120-cfm capacity rotameter can be operated in series with the orifice meter as a double check of the air flow rate. The pulsations from the blowers are absorbed by passing the air through both a small receiver and a muffler between each blower and manifold. Because of this, accurate operation of the air flow meters is assured. The air piping is adjustable so that any type of diffuser can be installed in the tank to study its efficiency in the conventional position for spiral flow, or any other position.

In addition to the oxygenation rate studies, the various aeration devices have been studied photographically with high speed stereo-cameras utilizing an exposure of 1/5,000 and 1/10,000 sec and with a high speed

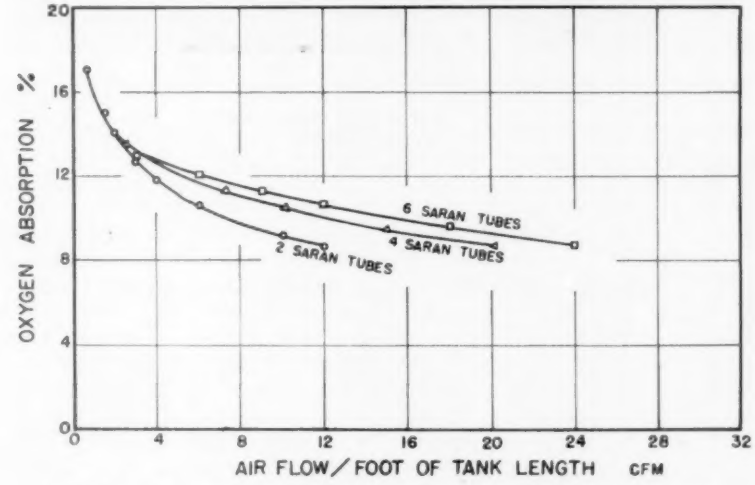

FIGURE 7.—Saran tube efficiency in terms of air flow per foot of tank.

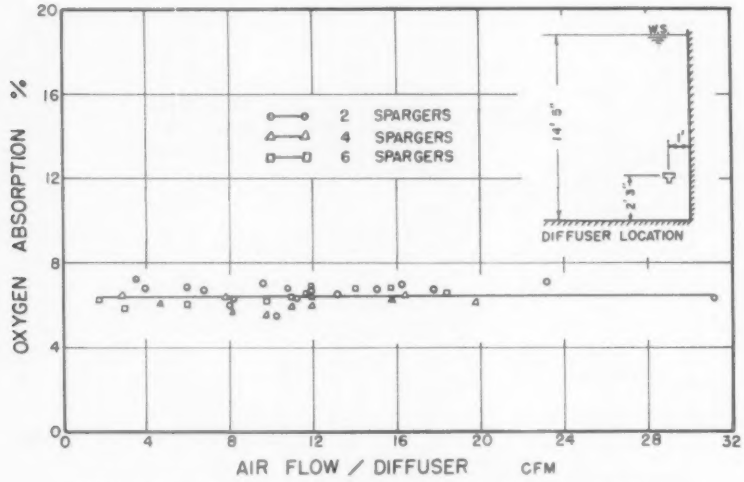

FIGURE 8.—Sparger efficiency in terms of air flow per diffuser.

movie camera operated at 600 frames per second. Although the photographic approach does not give quantitative data it does provide valuable and interesting information not available by direct observation. The stereo views are particularly valuable for indicating the true shape and form of the bubbles, and the high speed movie when viewed at 16 frames per second, appears to indicate that coalescence is not a problem in aeration tanks.

Saran Porous Media Diffusers

Standard Saran diffuser tubes 2 ft long by 3 in. outside diameter, wound 20 turns per in. were mounted in the aeration tank in the conventional spiral flow position. They were 2 ft above the bottom of the tank, and

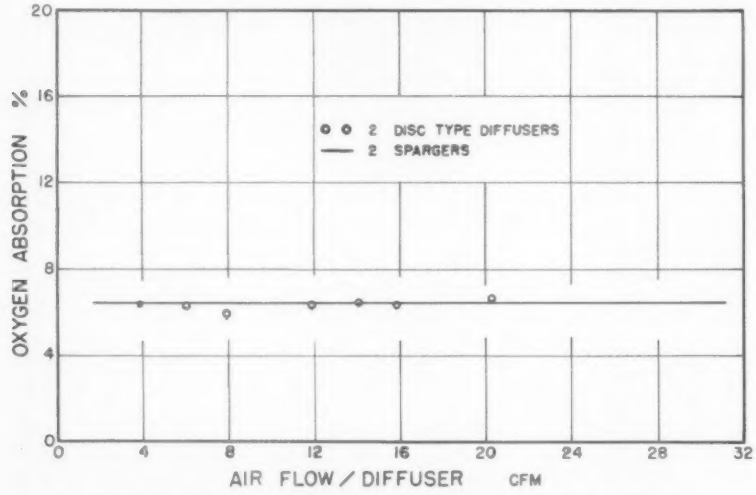

FIGURE 9.—Comparison of sparger and disc type diffuser efficiencies.

FIGURE 10.—Shear box bubble formation.

adjacent and normal to the 4-ft side of the tank. This setting provided a submergence of 12 ft 9 in. Oxygen absorption efficiencies were determined with two, four, and six tubes. The resulting efficiency curves are shown in Figure 6. At the rated capacity of 6 cfm per tube, the efficiency varies from 11.3 per cent with 1½ tubes per foot of tank to 12.6 per cent with on tube per 2-ft length of tank.

The same data have been replotted in Figure 7 to show oxygen absorption efficiency for various air flows per foot of aeration tank with 2, 4, and 6 tubes. It is apparent that for any given air demand per length of tank, increasing the number of tubes or reducing the unit rate of flow results in improved efficiency.

Sparger and Disc Type Diffusers

Spargers, each with four 5/16-in. diameter holes radiating horizontally from the center and 3-in. disc type diffusers (both as currently manufactured *) were checked for oxygen absorption efficiency and results are

* Manufactured by Chicago Pump Co., Chicago, Ill.

shown in Figures 8 and 9. Figure 8 shows the efficiency for 2, 4, and 6 spargers per 4-ft length of tank, and Figure 9 shows that spargers and disc type diffusers have similar oxygen absorption efficiencies. In each of these tests the diffusers were mounted in the conventional position, in a row 1 ft away from the wall and at an elevation such that the air discharged 2 ft 3 in. above the tank floor to provide 12 ft 3 in. of submergence.

It is apparent from these curves that at the rated capacity of 15 cfm these diffusers have an absorption efficiency of 6.4 per cent and this same efficiency applies for 2, 4, and 6 units per 4-ft length of tank. It is also apparent that these efficiency values hold for all flows except possibly for the slightly higher efficiency at the low flows per foot of tank.

Hydraulic Shear Diffusers

It was demonstrated that hydraulic shearing forces induced high oxygen absorption rates when the aeration column was confined within the single column of the two-column tank (Figure 3). This principle of hydraulic shear has been applied in two aeration devices and studied in the large full scale test tank.

The shear box is simply a sheet metal box 7 in. square and 12 in. deep, open at the top. In its elemental form the box is placed in the bottom of the aeration tank and air is discharged from an open pipe at the bottom center of the box. The air forms bubbles in the water and the resulting air-water mixture being lighter than water flows up and out of the shear box into the aeration tank. Water flows into the box to replace that which is carried out in the air-water mixture. The centrally rising air-water mixture and the downward moving replacement water all flowing within the limited space of the shear box create a turbulence or hydraulic shear which breaks the large bubbles emanating from the air inlet pipe into many smaller bubbles and develops the localized turbulence desired for increasing oxygenation efficiency (Figure 10). The 7- × 12-in. deep boxes were studied for air flows varying from 4 to 30 cfm per box. Other sizes and shapes were tried for the boxes but on the basis of this pre-

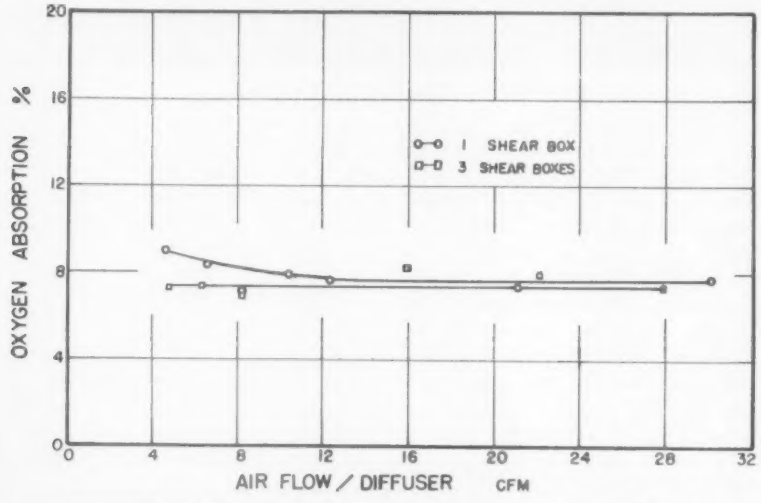

FIGURE 11.—Shear box efficiency in terms of air flow per diffuser.

liminary study the size selected appeared optimum.

In operation, the shear boxes are placed on the floor of the large test tank in a line one foot from the side wall, in the conventional position for spiral flow diffusers. Figure 11 shows the efficiency with one shear box and with three shear boxes per four-foot length of aeration tank.

The venturi-type diffuser was developed to utilize hydraulic shearing forces. It is a short venturi tube open at both ends, placed in a vertical position in the large aeration tank. In operation, air is discharged from a pipe at or below the throat of the venturi and the air-water mixture flows up through the venturi and discharges into the large aeration tank. The difference in the specific gravity of the air-water mixture in the venturi and its discharge pipe as compared to the specific gravity of the water surrounding the venturi provides the necessary head for creating the flow through the venturi, as in any air lift pump. However, in the venturi diffuser the energy is all expended in accelerating the water through the venturi since there is no static lift. At the throat of the venturi where velocity is maximum, the shearing forces within the flow break the large bubbles issuing from the air pipe into many small bubbles similar to those produced by the usual porous media diffuser. In Figure 12 the lower picture shows the large air bubbles issuing from an open pipe and the start of the bubble breakup in the throat of the venturi. The upper photo is the resulting air-water mixture issuing from the top of a 5-ft long venturi diffuser. Both views are at an air flow rate of 15 cfm in the 9-in. square unit.

Figure 13 is the efficiency curve for this same venturi diffuser. It was mounted with the inlet 10 in. above the bottom of the aeration tank and the tube was 5 ft long over-all. The point of air discharge had a submergence of 13 ft 7 in.

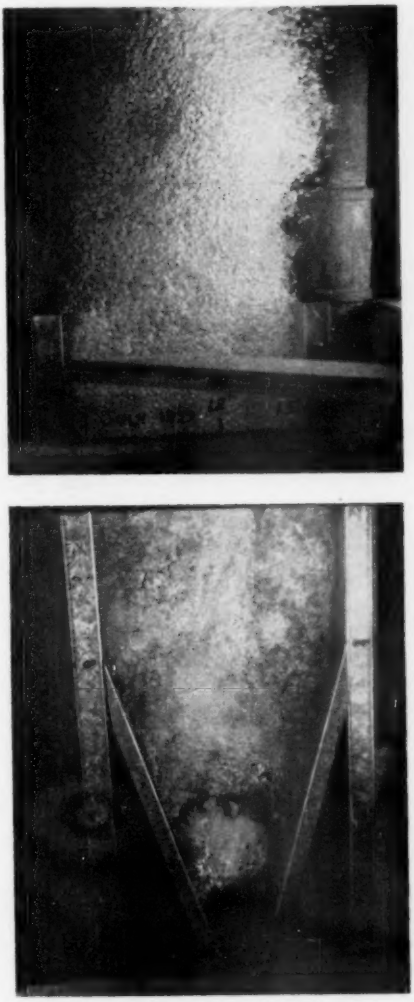

FIGURE 12.—Venturi-type diffuser bubble formation.

Conclusions

1. A "standard" method for evaluating oxygen absorption characteristics for various air diffusion devices is necessary, both to evaluate existing systems and to carry on studies for fur-

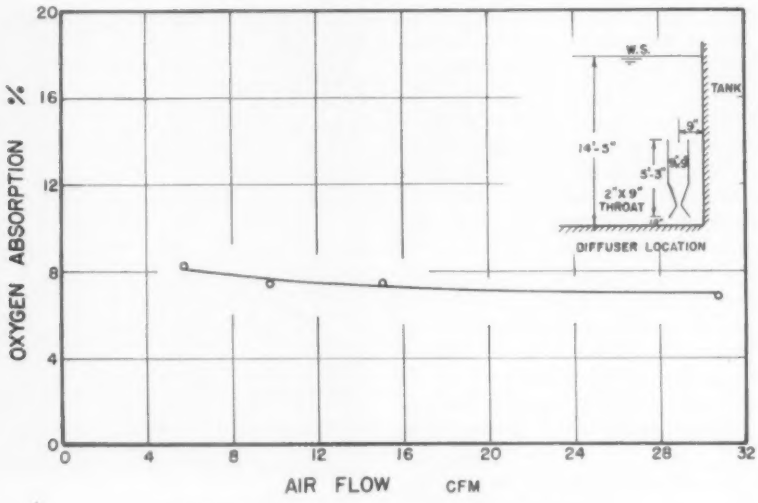

FIGURE 13.—Venturi-type diffuser efficiency.

ther improvements in absorption efficiencies.

2. The rate of oxygen uptake in any tank with a given diffuser is dependent on the absorbing media. It is shown that the absorption rate in sodium sulfite solution is dependent on the concentration of the catalyst and other variables. Sodium sulfite and biological suspensions are not satisfactory "standards."

3. The rate of oxygen uptake is dependent on the shape and size of the tank in which it is tested. The oxygen absorption rate of a specific diffuser tested in a confined column was 167 per cent of its absorption rate in a normally circulating flow of water.

4. It is recommended that for "standard" conditions, diffusers be tested for their oxygenation rate in a full-size tank in clear tap water at 20°C and zero dissolved oxygen. The oxygen absorption rates for air diffusion devices currently used in spiral flow aeration tanks are shown as tested in a tank 15.5 ft by 24 ft in cross section at "standard" conditions.

5. When the air-water mixture, at the point of air introduction into an aeration tank, is properly controlled, the bubble-liquid interfacial area and liquid turbulence are increased as a result of the development of hydraulic shearing forces. This results in increased oxygen uptake rates and improved aeration efficiency.

Acknowledgments

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The authors are indebted to David A. Long, Research Associate, and to Ward L. Akers and William R. Nicholas, Research Assistants, for their help in conducting these tests.

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OPPORTUNITIES IN MICHIGAN

The Michigan Department of Health has four openings for engineering personnel. These are tabulated below:

Section	No. Openings	Civil Service Classification	Salary Range
Water	1	Sanitary Engr. III	$7,600–$9,354
Environmental Health	2	Sanitary Engr. III	$7,600–$9,354
Environmental Health	1	Sanitary Engr. II	$6,410–$7,646

Minimum qualifications for the Sanitary Engineer II are a bachelor's degree in civil, sanitary, or chemical engineering with a minimum of one year's experience.

Minimum qualifications for the Sanitary Engineer III are the same academic requirements and two year's experience.

The top of the salary range for each grade is reached in four annual increments.

Further details of job description and duties and applications may be obtained from Dr. Albert E. Heustis, State Health Commissioner, Michigan Department of Health, Lansing 4, Michigan.

CULTURE OF *OSCILLATORIA* IN ORGANIC WASTES

By A. C. Gaur, W. O. Pipes, and H. B. Gotaas

Northwestern University Technological Institute, Evanston, Ill.

Extensive field observations and research studies (1)(2)(3) have shown that under proper conditions several species of algae may grow vigorously in wastewaters containing moderate concentrations of organic materials. In conventional stabilization ponds, aerobic bacteria depend to some extent upon oxygen produced photosynthetically by algae as an oxidizing agent in metabolic reactions by which they stabilize the organic wastes. Properly operated stabilization ponds can produce an effluent which is comparable in quality to that produced by any other biological sewage treatment process.

Some attention has been given to the possibility of employing stabilization ponds as units for algae production as well as waste treatment (4), thus reclaiming the nutritive value of some waste materials. It has been shown that small unicellular algae, particularly *Chlorella* and *Scenedesmus*, may reach significant populations (about 300 mg/l of algae cells) in stabilization ponds which are operated at short detention periods when the light intensity is great enough (3). To produce an acceptable effluent from the ponds operated as algae production units, it is necessary to remove the algal cell material. Because of the small size of *Chlorella* and *Scenedesmus*, the algal cells may be removed from the effluent only at considerable expense and the technical difficulties of handling and drying the harvested algae at a reasonable cost are great.

This investigation was undertaken as an attempt to determine if there are filamentous algae which would grow satisfactorily with bacteria in water containing organic wastes and which might be harvested more economically than the algae already studied. A review of the literature indicated several genera of filamentous blue-green algae and a few filamentous green algae do occur in highly polluted waters.

Preliminary experiments with the filamentous green algae *Ulothrix* and *Hormidium* were disappointing, although some growth of *Hormidium* was obtained. However, a strain of *Oscillatoria*, a filamentous blue-green alga which grows quite well in sewage, was obtained and it was decided to proceed with experiments to determine the growth characteristics of this alga in sewage and other organic wastes.

Methodology

The particular strain of *Oscillatoria* employed in this investigation was isolated from a sample of domestic sewage collected at the Northside sewage treatment plant of the Chicago Metropolitan Sanitary District. The *Oscillatoria* growth was observed in the sewage after it had been sitting in the laboratory in a covered container for a period of three weeks.

The experimental growth studies were conducted in vertical glass tubes having a diameter of 60 mm and a liquid volume of 1 l. Figure 1 is a photograph of the growth units. Mixing of the growth substrate was accomplished by bubbling air upward from the bottom of the growth units. The culture received a light intensity

at the surface of 100 ft-c which was supplied by fluorescent lights. The operation of the growth unit was intermittent. Feeding of nutrient solution and removal of effluent was accomplished manually twice a day in amounts depending on the desired detention period. The temperature of the cultures was maintained at 27°C ± 2°. The collected effluents were analyzed for pH, total alkalinity, COD using the dichromate reflux method, organic and ammonia nitrogen, and, in some experiments, BOD.

Each culture initially was seeded with 250 mg dry weight of *Oscillatoria*. The algae were allowed to accumulate in the growth units and were harvested after the 14-day experimental periods. The total volume of the settled algae was measured and a known volume of algae was dried to constant weight at 103°C. The algae were analyzed for protein, ether extract, and ash contents. All determinations were carried out according to the procedures described in "Standard Methods" (5).

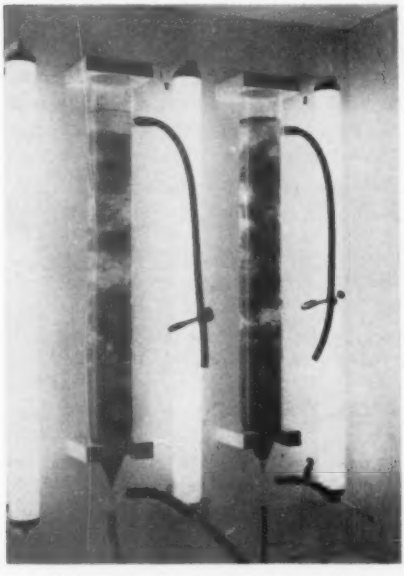

FIGURE 1.—Growth culture units.

TABLE I.—Analysis of Sewage and Skim Milk Media

Constituent	Concentration (mg/l)	
	Sewage	Skim Milk*
Total solids	448	1,166
Ash	—	88.8
Total nitrogen	19.4	57.2
Organic nitrogen	5.3	55.2
Ammonia nitrogen	14.0	2.2
BOD	—	630
COD	—	1,050
Lactose	—	509
Alkalinity	220	160
pH	7.8†	6.8†

* 1 g/l skim milk in tap water.
† pH units.

The experiments were carried out in four series, each series consisting of three cultures having different detention periods (2, 4, and 6 days). The growth medium employed in Series I was domestic sewage, in Series II it was dried skim milk at a concentration of 1.0 g/l in tap water, in Series III it was 1.0 g/l skim milk in tap water plus 10 ml sewage per liter, and in Series IV it was the skim milk at a concentration of 1.5 g/l. The analysis of the sewage and skim milk which was employed as the growth medium are presented in Table I.

Results

The data showing growth rate and yield of *Oscillatoria* for different detention periods in a municipal sewage and in the skimmed milk media which simulate a dairy waste are shown in Table II. The third column of the table is the concentration of the algal material which settled from the culture at the end of the 14-day growth period less the concentration of *Oscillatoria* which was used as an inoculum. The fourth column is the growth rate of the *Oscillatoria* per liter per day and was obtained by dividing the values given in the third column by 14 days. The last column

TABLE II.—Rates of Production of *Oscillatoria* in Organic Wastes

Exp. No.	Detention Period (days)	Final Conc. in 14 Days (mg/l)	Growth Rate (mg/l/day)	Total Growth (mg/l)
I-1	2	2,276	163	325
I-2	4	2,150	154	614
I-3	6	1,014	72	434
II-1	2	2,088	149	298
II-2	4	1,646	118	471
II-3	6	1,395	100	598
III-1	2	2,080	149	297
III-2	4	1,681	120	480
III-3	6	1,470	105	630
IV-1	2	2,249	161	320
IV-2	4	1,795	128	513
IV-3	6	1,573	112	674

gives the total algal growth per liter of waste processed.

The results of Series I experiments using municipal sewage as the growth medium indicate that *Oscillatoria* grows well in municipal sewage and will produce a relatively high yield under conditions of low light intensity. The average daily yields are considerably lower than would be expected were the growth units operated continuously with an algal concentration close to that reached near the end of the 14-day period of the experiments. The NH_3-N in the sewage was removed by the algae as rapidly as it was added to the media after the first 2 or 3 days of growth. There was relatively little organic nitrogen available for conversion to NH_3-N by the bacteria. The limited quantity of ammonia partly accounts for the much greater growth rate for the 2- and 4-day detention periods and the larger yield of algae per liter of sewage processed for the 4-day detention period.

The data from Series II and Series III experiments indicate that *Oscillatoria* grow about as well in the skimmed milk as in the sewage. Apparently, the addition of 10 ml/l of sewage to the skimmed milk had little effect on the growth. Total production of algae in the Series II and III experiments was about the same as that of the Series I experiments even though the skimmed milk media contained a greater concentration of organic material and a much greater concentration of nitrogenous compounds. It is likely that the growth rate of the *Oscillatoria* in all three series was limited by the low light intensity at the surface of the culture and was not dependent on the concentration of any nutrient in the culture medium.

The data from Series IV experiments indicate only a 10-per cent increase in the total algae production although the concentration of nutrients in the medium was 50 per cent greater.

TABLE III.—Analysis of Cultures at the End of 14-Day Growth Period

Exp. No.	Culture Medium	Detention Period (days)	Algae Conc. (mg/l)	pH	Alk. (mg/l)	Organic-N (mg/l)	NH_3-N (mg/l)	COD (mg/l)	BOD (mg/l)
I-1	Sewage	2	2,276	9.1	100	0	0	—	—
I-2	↓	4	2,150	7.0	75	0	0	—	—
I-3	↓	6	1,014	7.8	59	0	0	—	—
II-1	Milk (1 g/l)	2	2,088	7.4	200	6.3	11.0	—	—
II-2	↓	4	1,646	7.4	180	8.0	6.1	—	—
II-3	↓	6	1,395	—	—	—	—	—	—
III-1	Milk (1 g/l) +sewage (10 ml/l)	2	2,080	7.9	140	6.2	12.1	200	23.6
III-2	↓	4	1,681	7.9	141	10.9	6.2	124	19.6
III-3	↓	6	1,470	7.7	160	8.4	4.5	—	—
IV-1	Milk (1.5 g/l)	2	2,249	7.3	200	11.7	13.4	232	34.1
IV-2	↓	4	1,795	7.8	198	16.8	12.8	182	21.8
IV-3	↓	6	1,573	—	—	—	—	—	—

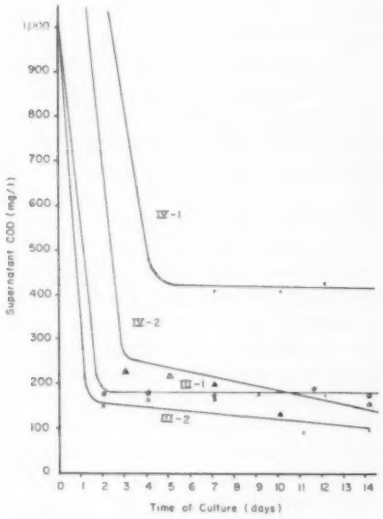

FIGURE 2.—Reduction in COD by *Oscillatoria*.

This also suggests that algal growth was limited by light intensity.

Also of considerable interest in this study was the degree of purification of the organic waste which accompanied the algae growth. Table III presents the analysis of the supernatant liquid which remained at the end of the 14-day growth period. By comparing the data in Tables III and I, it may be seen that the reduction in BOD for the cultures in which it was determined was about 97 per cent. Corresponding reductions in COD varied from 80 to 90 per cent.

The decrease in COD and BOD of the effluent from the cultures for Experiments III-1, III-2, IV-1, and IV-2 is shown graphically in Figures 2 and 3. The amount of oxidizable material in the effluent decreased rapidly during the first two or three days the culture was operated; thereafter it leveled off to an almost constant value. Apparently, it required only a short time for the proper bacterial population to be established.

Table IV presents the chemical composition of the harvested algae in terms of protein, ether-extracted fat, and ash as a per cent of total dry weight. The protein content of the sewage-grown algae is very low because of the limited nitrogen in the sewage medium and the ash content is high. On the other hand, the protein content of the algae grown on skim milk was high because of the NH_3-N which was available to the algae. It is assumed that carbohydrates account for the difference between 100 and the sum of the percentages for protein, fat, and ash. The amount of carbohydrate in the sewage-grown algae is larger than usually found.

Discussion

A steady-state condition with respect to algae population was not maintained; hence, the algae yields are average values. Studies of yields of *Chlorella* and *Scenedesmus* have been made under steady-state conditions and higher light intensities. Oswald *et al.* (4) report for *Chlorella pyrenoidosa* in sewage under light intensity of 1,200 ft-c maximum yields of 115 mg/l in a growth unit similar to the one used in

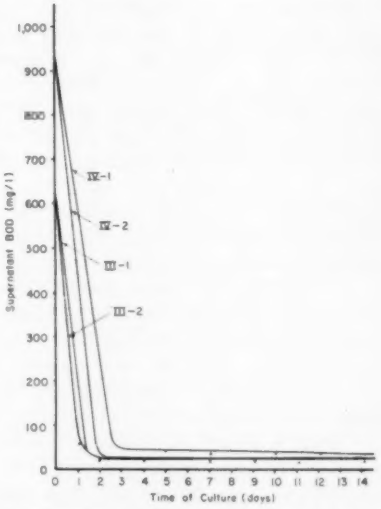

FIGURE 3.—Reduction in BOD by *Oscillatoria*.

TABLE IV.—Chemical Composition of *Oscillatoria*

Medium	Protein (%)	Ether-Extracted Fat (%)	Ash (%)
Sewage	21.5–25.0	19.0	17.0
Skim milk	40.9–44.4	18.3–25.3	6.6–6.2

these experiments. They also found a maximum yield of 85 mg/l for *Euglena gracilis* growing under a light intensity of 1,200 ft-c (6). In other studies, Oswald and Gotaas report yields of over 300 mg/l for *Chlorella* and *Scenedesmus* in sewage having a BOD of over 300 mg/l with a light intensity at 900 ft-c and a detention period of 3.25 days. When it is considered that the light intensity for the experiments reported herein was only 100 ft-c at the surface of the culture tube, that the older algae were allowed to accumulate in the growth unit, and that the culture was started with a small algae inoculation, yields of 150 mg/l/day compare very favorably with those for unicellular algae. Also, yields up to 600 mg per liter of waste processed are excellent for the low light intensity.

Observations of reduction in COD and 5-day BOD in the skimmed milk culture show 80 to 90 per cent removal of COD and 95 to 97 per cent removal of the BOD. These results are a little higher than those obtained by Oswald *et al.* in the effluents from sewage cultures of *Chlorella* (4). BOD is reduced by the action of aerobic bacteria and a high degree of removal would be expected for the detention periods used since the cultures were aerated continuously. However, activated sludge type of floes were not observed. Apparently the bacteria were growing either more-or-less dispersed or attached to the algae cell surfaces.

Figure 4 shows the algae after five minutes' settling in the culture tubes. The settled algae filter easily to a cake containing less than 80 per cent moisture. In contrast to small unicellular algae, *Oscillatoria* do not present filter-clogging problems.

The unicellular algae in a mixed stabilization pond do not settle, but remain suspended in the liquid. Some coagulation and sedimentation of organic matter does take place. The bottom material continues to decompose and the decomposition products become available as food for the algae. Hence there are three phases of biological growth in the pond: (*a*) stabilization of organic matter on the pond bottom, (*b*) stabilization of organic matter by bacteria suspended in the liquid, and (*c*) algae growth in the part of the liquid penetrated by light. If *Oscillatoria* or another filamentous algae will grow suspended in the liquid they could be harvested by a simple mechanical operation and filtered to a cake fairly economically. Even if the specific growth rate of the filamentous algae were less than that of unicellular algae, maintenance of a large concentration of algal cells in the pond might make possible a larger yield per unit

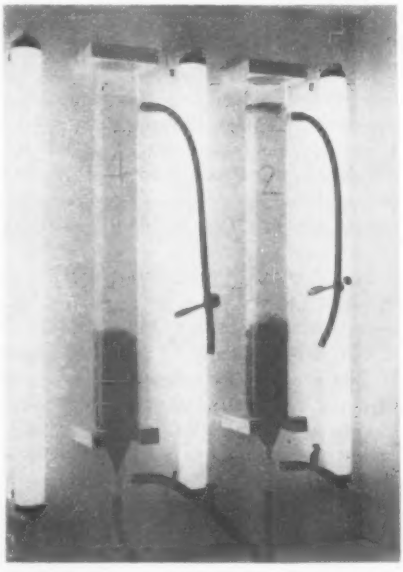

FIGURE 4.—*Oscillatoria* after five minutes' settling.

area of pond surface than can be achieved with unicellular algae.

Another problem of a practical nature is whether the smaller algae would overgrow and shade out *Oscillatoria*. If *Oscillatoria* or other filamentous algae can be made to remain suspended in the pond liquid, the detention time of the pond might be made sufficiently short so that the small free-floating algae cannot build up a high enough concentration to prevent the growth of filamentous algae. Whether protozoan organisms which would feed on the algae could be controlled in a pond when the algal mass was more or less stationary is not known.

It is possible that a plant which does not root, but will float as stationary, branching, thread-like networks and through which the liquid can flow, could provide the most efficient pond-operating conditions from the standpoints of supplying oxygen, yield of algae, and harvesting.

Much further study will be necessary to judge the characteristics and qualities of *Oscillatoria* to meet these conditions. This work represents a preliminary investigation. Perhaps others will explore this organism further or find other organisms which offer more promise for achieving an effective method for supplying oxygen for aerobic stabilization and reclamation of valuable nutrients in wastewaters.

Summary and Conclusions

1. A strain of *Oscillatoria* grew well under laboratory conditions in sewage and other organic wastes producing yields under limited light conditions which appear to be in the same range of magnitude as those found for *Chlorella*, *Scenedesmus*, and *Euglena*.

2. Under the conditions of *Oscillatoria* growth with aerobic bacteria, 95 to 97 per cent of the 5-day BOD is removed. Reductions of 80 to 90 per cent of the COD were obtained.

3. The algae contained 40.9 to 44.4 per cent protein, 13.3 to 25.3 per cent ether-extracted fat, and 6.0 to 6.2 per cent ash when grown in milk waste media.

4. Based on the results of this brief study, *Oscillatoria* appears to have characteristics which make it potentially satisfactory for growth in wastewater stabilization ponds. Extensive further studies are necessary to determine more adequately its characteristics under a variety of conditions.

Acknowledgments

This research was supported by grants-in-aid from the Research Corporation of America and the National Institutes of Health.

The early identification of *Oscillatoria* found in the containerful of Chicago sewage was made by Dr. H. L. Tiffany of the Department of Biological Sciences, Northwestern University.

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EXPERIMENTS ON THE HIGH-RATE ACTIVATED SLUDGE PROCESS

By Dietrich Kehr and Wilhelm von der Emde

Hannover, Germany

The Department of Sanitary Engineering of the College of Technology (Technische Hochschule), Hannover, Germany, in the course of the last few years has been conducting experiments on pilot plants and in large-scale plants on high-rate activated sludge processes. Some of the experiments are described in this report.

PILOT-PLANT STUDIES AT KASSEL

Kassel has a population of about 200,000, and out of that number about 150,000 live within the area of the drainage system tributary to the main sewage treatment plant. This plant was constructed in 1898 and since then has been expanded. Sewage is treated in a sedimentation tank with an average retention period of one hour.

The effluent is discharged to the river Fulda, which combines with the river Werra to join the Weser at a point about 12 miles downstream from Kassel. The Fulda is heavily loaded with wastes from Kasssel which have received only primary sedimentation. The Municipal Council wants biological treatment of the sewage and Dr.-Ing. Karl Imhoff has suggested in his report that a high-rate activated sludge process combined with primary treatment could achieve an over-all BOD removal of 75 per cent. The Council, to study the partial biological treatment of the sewage, constructed a pilot plant and the Department of Sanitary Engineering (Institut für Siedlungswasserwirtschaft) of the College of Technology, Hannover, was asked to undertake experiments.

Scope of Experiments

The following aspects were considered in the pilot-plant experiments:

1. What would be the volume of tank necessary for aeration?
2. What would be the most suitable system of aeration?
3. What would be the power consumption?
4. What would be the expected BOD removal?

The Pilot Plant

The layout of the pilot plant is shown in Figure 1. It was divided into three parallel units so that each could be run simultaneously. Simultaneous operation was considered essential because the sewage concentration at times varied enormously. The desired quantity of effluent from the primary plant was supplied to the distributor tank by an automatic suction pump, and with the help of regulators, the desired quantity of sewage could be fed into any unit.

The three aeration tanks were of equal capacity, 708 cu ft each. The cross sections of the tanks are shown in Figure 2. Secondary settling tanks each had a capacity of 885 cu ft and a surface area of 95 sq ft. The aeration tanks and secondary settling tanks were constructed from steel sheets and were transportable.

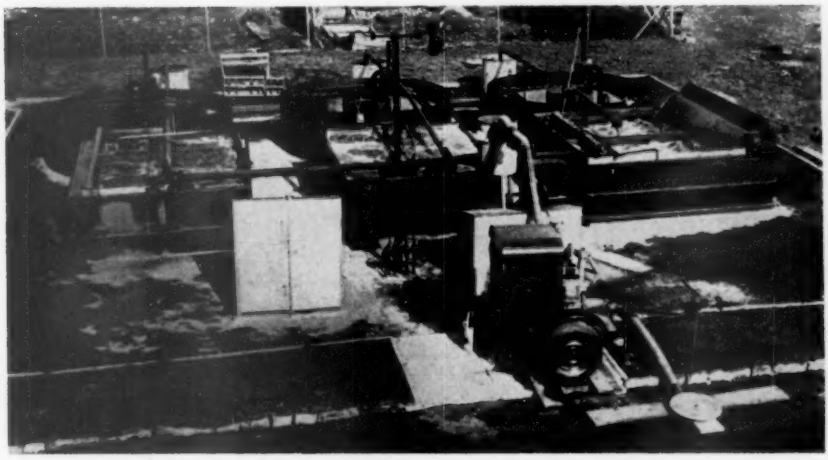

FIGURE 1.—Pilot plant at Kassel.

Experiments

Experiments were started in September 1955. In aeration tank No. 1, porous cones (Schumacher medium pores), large-bubble aeration as designed by Passavant, and medium-sized porous lattice as designed by Inka were built in succession and tried. In aeration tank No. 2, Schumacher porous pipes (of large, medium, and fine porosity) were tried. Aeration tank No. 3 was run consistently with a Kessener-brush aerator, using the type of brush 20 in. in diam as designed and developed by Messrs. Passavant & Co., with angle-iron brushes and Pasveer's guard attachment.

To vary the supply of sewage for the same volume of secondary settling tanks, the capacities of the aeration tanks had to be changed. Accordingly, during the latter half of the experiments, each aeration tank was reduced to half size and the one with the Kessener brush attachment was also reduced in size with the help of a partition. Toward the end of the experiments, two secondary settling tanks in series were connected to an aeration tank with the idea of working with double the quantity of sewage.

In the course of the experiment, the quantity of sewage varied from 62 to 67 gpm. The retention period in the

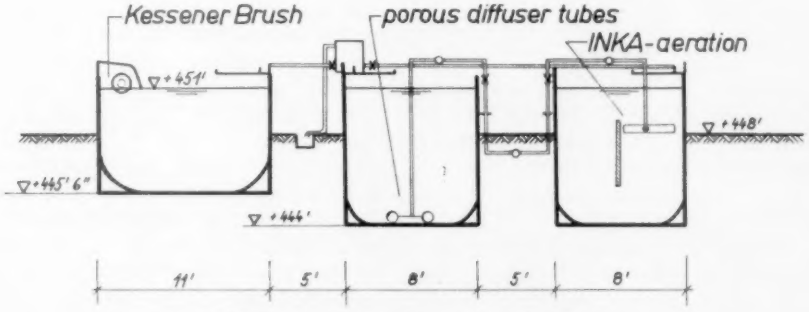

FIGURE 2.—Cross section of aeration tanks.

TABLE I.—Typical Results of Pilot-Plant Tests *

Test No.	Hydraulic Load (gpd/cu ft)	BOD					Mixed Liquor SS (mg/l)	SVI	COD†			Energy Requirements	
		Infl. (mg/l)	Effl. (mg/l)	Red. (%)	Load (lb/day/1,000 cu ft)	Load (lb/lb SS)			Infl. (mg/l)	Effl. (mg/l)	Red. (%)	(kwhr/day/1,000 cu ft)	(kwhr/lb BOD removed)
					(a) Brandol 60 Diffuser Tubes								
1	202	141	19	87	238	0.9	4,200	80	237	73	69	48	0.232
2	300	141	25	82	350	1.4	3,900	65	236	86	64	62	.216
3	291	84	12	86	206	0.7	4,800	70	193	65	66	68	.382
					(b) INKA								
1	404	138	34	75	462	2.1	3,600	61	244	103	58	79	0.228
2	418	99	23	77	350	1.4	3,900	61	198	83	58	82	.304
3	456	141	38	73	537	2.1	4,200	51	237	98	59	83	.210
					(c) Kessener Brush								
1	583	84	21	75	412	1.1	6,000	43	193	77	60	116	0.376
2	600	141	34	75	644	2.7	4,200	53	236	101	57	110	.208
3	687	141	41	71	812	2.5	5,100	42	237	106	55	110	.191

* Sept. to Oct., 1956.
† Permanganate method.

secondary settling tanks varied from 1.2 to 1.8 hr. The surface load in the settling tanks varied between 900 and 1,450 gpd/sq ft and the speed at the critical cross section varied between 0.13 and 0.22 fpm.

In each aeration unit a BOD loading of about 154 lb/day was tried. Assuming a per capita BOD value for settled sewage of 0.08 lb/day, this loading works out to a population equivalent of about 2,000.

Results

Some of the useful results obtained have been set out in Table I. A discussion of the results is presented under the separate headings below.

Sewage

The sewage of Kassel is of the dilute domestic type and the BOD during the first month of the experiments was about 140 mg/l after primary treatment. During the summer months, the BOD fell to 60 mg/l due to continuous rainfall and consequent infiltration. In the months of September, October, and November, 1956, the BOD rose to 160 mg/l, averaged over weekly periods.

The hydraulic load (1) in the individual tanks varied within wide limits. The relationship between the hydraulic load and the efficiency of purification can be judged easily from Figure 3.

In the course of the experiments it became obvious that the final choice of aeration system lay among fine-bubble aeration with Schumacher's porous pipes (diffuser tube Brandol 60), Kessener brushes, and the Swedish INKA methods; accordingly, only the details of these three have been taken into consideration for graphical representation. In the results, the weekly average values were taken into account with three readings taken daily. These results showed that a BOD reduction of 80 to

90 per cent could be achieved with a hydraulic load of 300 gpd/cu ft and about 36 min of aeration. This figure for the hydraulic load was found applicable to all three aeration systems. When the hydraulic load was changed from 375 to 750 gpd/cu ft and the aeration period from 30 to 15 min, BOD reduction decreased to 70 per cent. The retention period here is the theoretical retention period and does not take into account the return sludge.

For an over-all BOD removal of 75 per cent, and estimating that the BOD reduction would be 20 per cent in the existing primary tanks, the efficiency of BOD reduction in the aeration tanks should be about 69 per cent. From the experimental results, this efficiency of purification could be obtained with an aeration time of 15 min.

In the latter half of the experiment, for a period of only three weeks, a hydraulic load of 1,120 gpd/cu ft with 10 min aeration was tried, the result being 60 per cent BOD reduction.

It appears that for design purposes a hydraulic load of 300 gpd/cu ft with an aeration time of 36 min would result in at least 70 per cent efficiency. If necessary, a BOD reduction of 85 per cent could be achieved.

The reduction in COD as determined by the permanganate method varied between 50 and 70 per cent, and at a hydraulic load of 300 gpd/cu ft, the average reduction was 65 per cent.

The relationship between the BOD load and reduction is shown in Figure 4. For a BOD load of 375 lb/day/1,000 cu ft, the efficiency of purification averaged 75 per cent. Even at a BOD load of 900 lb/day/1,000 cu ft there was a reduction of over 70 per cent. Thus for the design of the full-scale plant for Kassel, a BOD loading criterion of 425 lb/day/1,000 cu ft was selected.

Activated Sludge

The relationship between BOD load in terms of the sludge solids in the mixed liquor and BOD reduction is depicted in Figure 5. It shows that when the loading was 1.0 lb of BOD per lb of suspended solids, the BOD reduction was 85 per cent, and when the loading was 1.5 lb of BOD per lb of SS, the reduction was 80 per cent. Even when the loading stood at 2.5 lb of BOD per

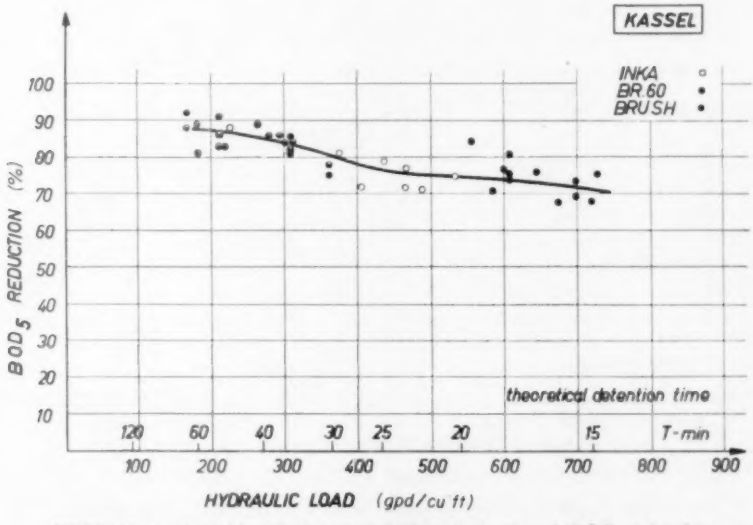

FIGURE 3.—Relationship between hydraulic load and BOD reduction.

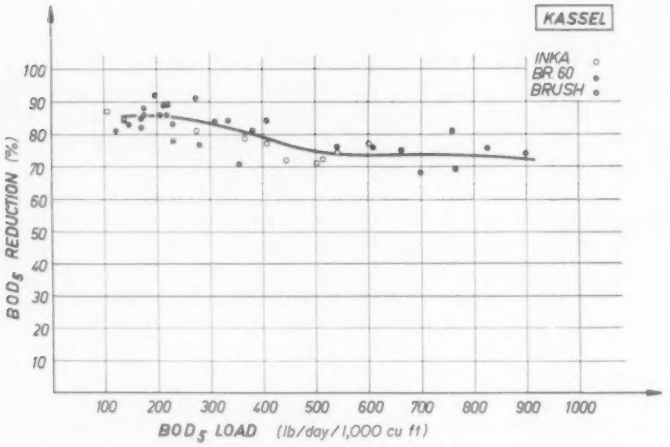

FIGURE 4.—Relationship between BOD load and BOD reduction.

lb of SS, the reduction only decreased to 75 per cent. Therefore, to obtain an over-all efficiency in the whole plant of 75 per cent, the loading should be 2.5 lb of BOD per lb of SS. For the maximum BOD load of 470 lb/day/1,000 cu ft, it is necessary to maintain the mixed-liquor solids at 3,000 mg/l.

The concentration of the suspended solids in the aeration tanks varied mostly between 3,000 and 5,000 mg/l and between 7,000 and 10,000 mg/l in the return sludge. The organic matter present in the dry sludge was 65 to 75 per cent. The settling qualities of the sludge as determined in terms of sludge volume index and the relationship between the index and BOD load are

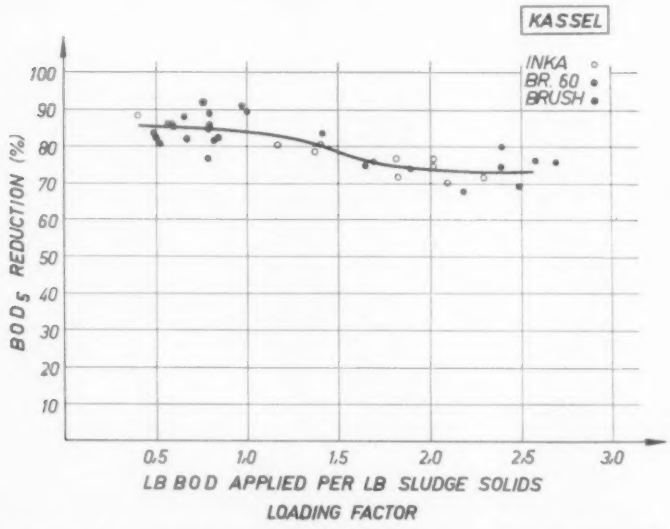

FIGURE 5.—Relationship between BOD load in terms of suspended solids and BOD reduction.

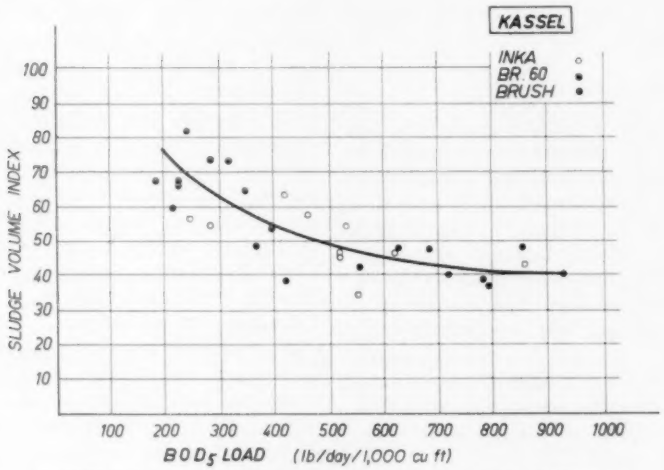

FIGURE 6.—Variation of sludge volume index with BOD load.

shown in Figure 6. From the diagram it is apparent that as the load increased, the sludge volume index decreased and the sludge quality deteriorated.

Biologically, the activated sludge throughout most of the experiments contained many colonies of protozoa, especially stalked ciliates such as *Vorticellae* and *Opercularia*. Only with the highest BOD loading did flagellates replace the ciliates, and this despite excess oxygen being present. With a short retention period of 20 min, the sludge had a predominantly bacterial makeup. At a low BOD loading, a decrease in the numbers of the stalked ciliates and an increase of the free-swimming forms and flagellates were observed whenever a low dissolved oxygen content was maintained for a period in the aeration tanks.

With only traces of dissolved oxygen, bacteria were observed in the sludge. Numerous thread-like organisms were also obvious, but these never became the main flora. Samples of bulked sludge did not appear to contain any *Sphaerotilus* although this organism was previously thought to cause bulking.

The only microscopic difference found between the different types of aeration tanks, when working with only traces of dissolved oxygen, was that the sludge from the Kessener tank contained thread-like organisms which were shorter than those in the other tanks. It is probable that this was caused by the mechanical agitation of the rotating brushes.

The rate of sludge production varied considerably; excess sludge varied from 50 to 150 lb/100 lb of BOD load. The average rate of production can be considered to be between 80 and 100.

Oxygen

Unless the necessary amount of oxygen was supplied, the efficiency of purification definitely decreased. In the pilot plant at Kassel, the DO was determined with an Oxydimeter (2). With average DO of 2 mg/l or more, no advantages could be detected for higher oxygen levels.

The DO usually varied between 3 and 5 mg/l. This was true particularly in the period from August to November, 1956, when the aeration tanks were run with a relatively high load. For a larger plant, it may be expedient to maintain a minimum oxygen content

of about 2 mg/l to minimize energy consumption.

Operation of the Plant

The short detention period in the primary sedimentation tank (about 45 min during daytime) did not interfere with the operation of the pilot plant. In view of the adequate biological purification, a bigger primary sedimentation tank was not considered necessary. At periods of high flow in the river Fulda, sewage was treated without primary sedimentation. But immediately a notably lower sludge volume index occurred and it was, therefore, thought expedient to waste some of the activated sludge.

Sewage was distributed evenly throughout the length of the tank by incremental feeding, and due to the short length of the pilot-plant tanks, the flow was immediately well mixed. It was observed that there was less chance of short-circuiting with this method of biological purification. Only a few minutes contact was sufficient to carry out the treatment process.

The amount of return sludge was varied between 20 and 150 per cent. No influence on the results was detectable by varying the quantity of return sludge.

The most important aspect of the operation seemed to be the method by which oxygen was introduced into the sewage. Turbulence varied according to the aeration system used (fine-bubble, course-bubble, or surface aeration) and the amount of oxygen absorbed. For the high BOD load at Kassel, the supply of oxygen had to be high which meant that the fine-bubble aeration created intensive turbulence. The increased mechanical agitation necessitated in the Kessener-brush tank compared favorably in this respect with the INKA method which gave medium-sized, flat-type bubbles and generated considerable turbulence in the upper half of the aeration tank.

It was observed that for the same load and oxygen content, the BOD removal was nearly the same in all three tanks. The differences were all within the limits of accuracy of the chemical tests. During the major part of the experiments, the different systems of aeration were not run with equivalent loadings. The results plotted in Figures 3 and 4 of hydraulic load against BOD load show that the Kessener-brush plant was run at the highest load and the fine-bubble aeration plant at the lowest. These values were obtained under conditions of pilot-scale experiments and cannot, therefore, be used to determine limits of efficiency of any individual aeration system. With a higher loading on the aeration tanks of the INKA and fine-bubble systems, results comparable to the Kessener system probably would have been produced. In that case an amount of oxygen similar to that supplied the Kessener plant would have been required. The correctness of this assumption with regard to the INKA system was proved in the last few weeks of the tests when purification comparable to the Kessener system was achieved with a retention period of only 10 min. From the engineering point of view, the aeration systems described above are all equally applicable for the conditions prevailing in Kassel.

The amount of oxygen supplied each of the aeration tanks at the same load and nearly the same sludge content was determined approximately with the help of the Toedt Oxydimeter. To obtain the same amount of dissolved oxygen in all aeration tanks, it was necessary to increase the air supply to the Passavant porous-plate, large-bubble, aeration system 1½ times over that for fine-bubble aeration (Brandol 60). Coarse-grained Brandol 300 diffuser tubes for large-bubble aeration were found to give an oxygen transfer similar to the Passavant porous plates. Aeration by INKA lattice, Brandol 60 tubes, and Kessener brushes produced

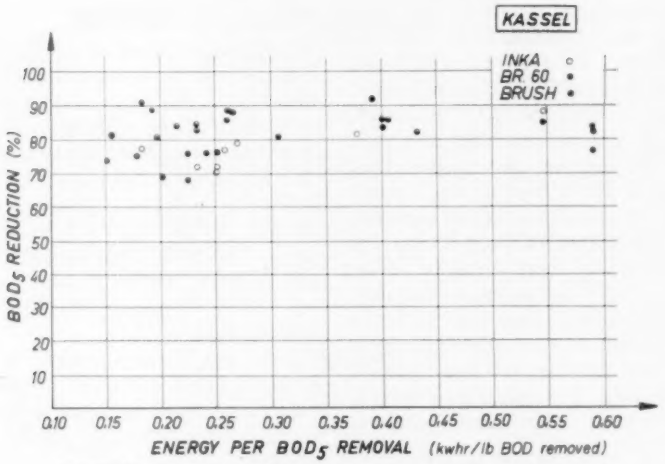

FIGURE 7.—Relationship between energy requirement and BOD reduction.

approximately the same amounts of dissolved oxygen for the same energy consumption. Differences were within the limits of accuracy with which the plant would operate in the future.

Energy requirements per lb of BOD removed for various BOD efficiencies are shown in Figure 7.

It shows that beyond a power consumption of 0.18 kwhr/lb of BOD removed there was no increase in BOD removal. For the value to exceed 0.27 kwhr/lb of BOD removed, particularly dilute sewage is needed. The average value of energy consumption is about 0.23 kwhr/lb of BOD removed. If a larger plant is to be run with an average dissolved oxygen content of 2 to 3 mg/l, energy consumption generally would amount to 0.14 to 0.18 kwhr/lb of BOD removed.

The variation of BOD reduction for

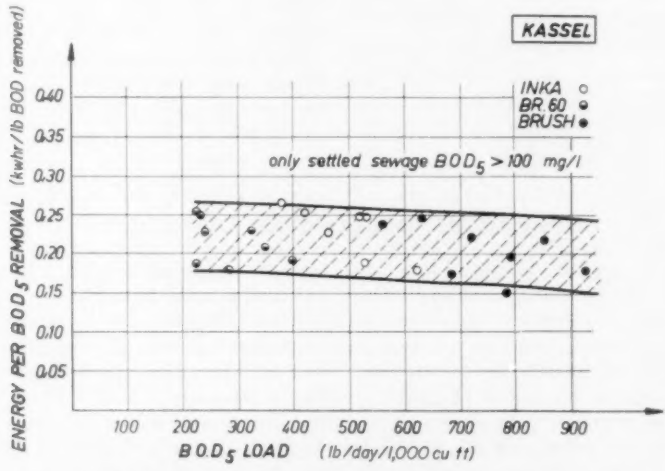

FIGURE 8.—Variation of energy requirement with BOD load.

the same energy consumption was caused by the variation of sewage and BOD loading. In Figure 8, BOD load is plotted against the energy consumption. Only values of BOD of incoming sewage greater than 100 mg/l are plotted. It appears that there is no influence of BOD load on the energy consumption in terms of BOD removal. In fact, the higher BOD loads did not have a greater energy demand; on the contrary, they had a lower demand. The diagrams show that with both the Kessener brush and the other two aeration systems similar low consumption figures result. The readings obtained with the Toedt Oxydimeter confirm these results.

In consideration of the costs involved in the three different systems, Kessener-brush aeration was found particularly suitable for the specific conditions of the sewage of Kassel. As the ground-water table is at a high level (it practically rises to the surface of the secondary settling tank), tanks should not be excavated deeply; therefore, the Kessener-brush method, requiring a flat, rectangular tank having shallow depth and greater width, would be more suitable. During the last of the tests, a rectangular tank having a width of 11 ft and depth of 3 ft 4 in. was designed. The Kessener-brush system also is considered the most suitable device because of its assurance of operation and the ease with which it can be adjusted to supply oxygen up to the demand of the sewage.

Summary

1. The introduction of a high-rate activated sludge process has been found suitable for the future biological treatment of the waste of Kassel. From the experimental results, it has been found that a hydraulic loading of 300 gpd/cu ft with an average retention period of 36 min can be used as a basis for design. The BOD load should be about 425 lb/day/1,000 cu ft when considering a BOD value of 170 mg/l for the settled sewage.

2. It has been proposed to install Kessener-brush aeration with angle-iron pieces made by Messrs. Passavant & Co. and with Pasveer's guard attachment.

3. The power consumption is calculated to be 0.20 kwhr/lb of BOD removed when the efficiency is about 75 per cent. An energy consumption of about 0.16 kwhr/lb would work out to 0.5 w/cap.

4. The results of the experiments show that the BOD reduction in the biological treatment reaches 69 per cent which safely represents 75 per cent over-all removal including primary sedimentation. With proper operation of the plant, a sewage load of 300 gpd/cu ft on the activated sludge units would give a BOD removal efficiency of 85 per cent and the total removal of the entire plant would then increase to 90 per cent. This means that the treatment plant could even deal with higher quantities of sewage and still achieve proper purification.

OPERATION RESULTS AT DETMOLD

Detmold and its surroundings are the center of the north Germany wood and furniture industry. Although the furniture industry itself does not discharge any water-borne waste, the plywood industry does. Near Detmold, it has been discharging its wastes into the sewers of the town. With approximately 30,000 population connected to the town drainage system, the sewage has had a total population equivalent of about 100,000. The treatment plant constructed before World War I was still working with a separate unit for sludge digestion, but achieved only a low degree of purification. In view of the high strength of sewage, a biological step-process of purification was at

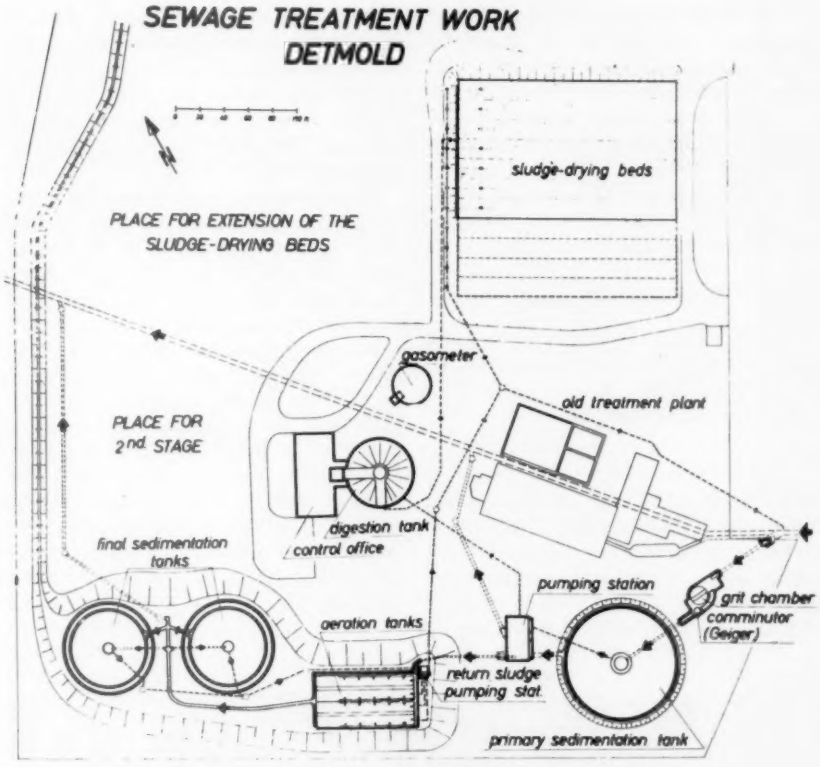

FIGURE 9.—Layout of Detmold sewage treatment plant.

first considered for a new plant. On the basis of the experiments undertaken by the Department of Sanitary Engineering, a two-stage aeration plant was considered for the design, and to prevent heavy impurities of the effluent polluting the river Werra into which the effluent flows, the construction of the new plant was urgently needed. The municipal board agreed to construct, as a first step, a treatment plant having short-retention biological treatment. The final full-scale biological plant was to be designed after further experiments.

Treatment Plant

The new sewage purification plant was constructed during the year 1955–56. Kehr and Schmidt-Bregas (3) described the plant and the layout is shown in Figure 9.

Sewage passes first through a circular grit-extractor and rotary screen. The effective capacity of the primary sedimentation tank is about 39,000 cu ft. The aeration tank has a capacity of 10,600 cu ft consisting of four 2,650-cu ft units with cross-sectional design developed by Pasveer (4). The aeration is by stainless steel Kessener brushes with Pasveer's guard attachment. The diameter of the brushes is about 16 in. and the surface speed is maintained at about 550 fpm (128 rpm).

The effective length of the brushes is about 276 ft and sewage is distributed

over ¾ of the tank length by means of dropping channels as designed by Schmitz-Lenders. Two circular tanks having a total capacity of about 49,500 cu ft serve as secondary settling tanks. Sludge is returned to the aeration tank by a screw conveyor. Figure 10 shows the arrangement of the aeration tank together with the Kessener brushes and Pasveer's guard attachment.

Operating Results

The biological section of the new plant at Detmold started operating during June 1956. Results set out in Table II are discussed below.

Sewage

The strength of the sewage varied during the period because of the diluting effect of the combined system.

The waste from the plywood factories can be heavily contaminated with impurities (BOD of 3,000 mg/l; $KMnO_4$ COD of 15,000 mg/l), but due to the diversity of work in the factories, the strength of the wastes is subject to large variation. Thus, the BOD of the incoming sewage to the biological plant has been known to vary between 50 and 550 mg/l.

Sewage going to the biological plant varied in quantity from 0.16 to 0.32 mgd. This constitutes a hydraulic load between 150 and 300 gpd/cu ft. In Figure 11, the average values of several tests have been plotted and the relationship between hydraulic loading or retention period and BOD reduction is shown. Experimental results have been computed and grouped according to loading (150 to 187 gpd/cu ft, 187 to 224, and 224 to 300) and BOD of the incoming sewage (100 to 200 mg/l, 200 to 300, and greater than 300). BOD values less than 100 mg/l were not considered.

It has been observed that the BOD reduction depends more on the hy-

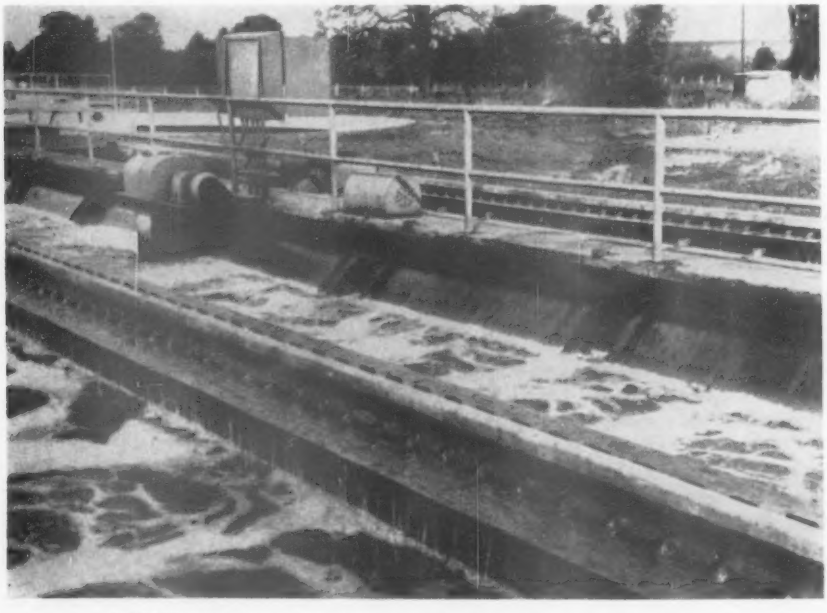

FIGURE 10.—Kessener brushes with Pasveer's guard attachment.

TABLE II.—Typical Operating Results Arranged in Order of Increasing Hydraulic Load *

Hydraulic Load (gpd/cu ft)	BOD					Mixed Liquor SS (mg/l)	SVI	COD			Energy Requirements	
	Infl. (mg/l)	Effl. (mg/l)	Red. (%)	Load (lb/day/1,000 cu ft)	Load (lb/lb SS)			Infl. (mg/l)	Effl. (mg/l)	Red. (%)	(kwhr/day/1,000 cu ft)	(kwhr/lb BOD removed)
165	204	18	91	281	0.57	7,900	65	478	138	71	110	0.450
167	161	17	89	225	0.47	7,700	73	380	115	69	113	0.572
172	345	27	92	494	1.0	7,900	50	496	168	67	113	0.258
202	379	53	86	644	1.17	8,800	50	702	276	63	104	0.195
214	242	35	86	431	0.92	5,900	50	558	204	63	99	0.272
218	167	21	87	306	0.5	10,200	73	377	136	64	88	0.335
247	224	30	86	455	0.82	8,900	44	471	156	67	91	0.244
253	147	21	84	312	0.45	11,200	40	357	122	65	83	0.312
277	141	23	84	325	0.58	8,900	42	329	127	59	88	0.326

* Sept. 1956 to Apr. 1957.

draulic load than on the strength of the sewage. According to the existing conditions at Detmold, it has been found that a hydraulic load of 180 gpd/cu ft or a retention period of 60 min is the optimum for operation. This gives, on the average, a 90 per cent BOD removal during biological treatment. With a loading of 224 to 300 gpd/cu ft, the removal is still 85 per cent even though the retention period is then only about 40 min. The graphical representation in Figure 12 (where BOD load applied is plotted against BOD removal) shows even more clearly the influence of sewage loading. Curves have been drawn for hydraulic loads of less than 180 gpd/cu ft and loads between 180 and 300 gpd/cu ft. They are practically parallel to each other and show that the BOD load exerts little influence. For example, with a BOD load of 500 lb/day/1,000 cu ft, equivalent to a hydraulic load of 165

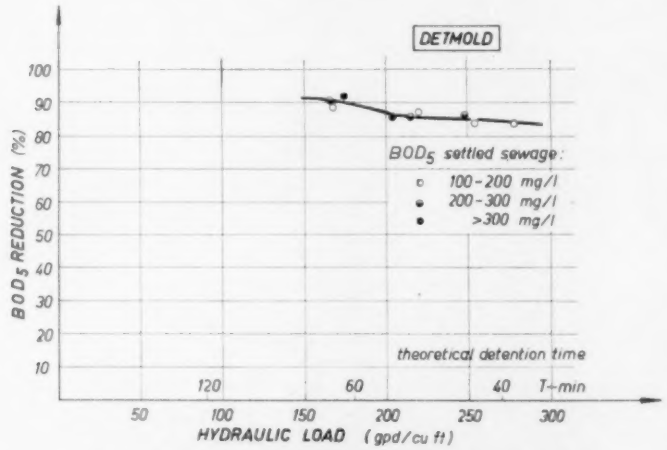

FIGURE 11.—Relationship between hydraulic load and BOD reduction.

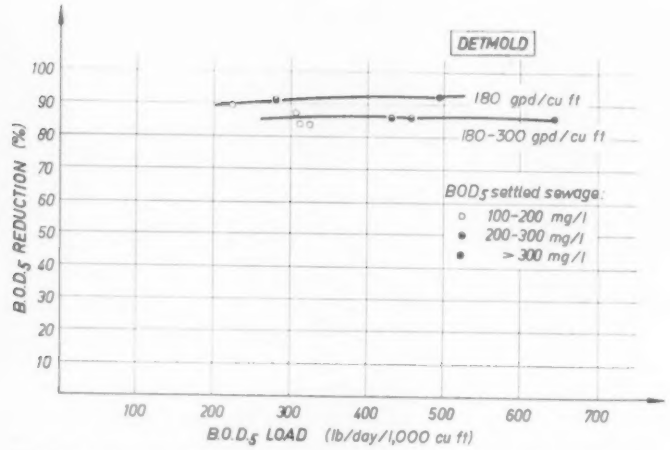

FIGURE 12.—Variation of BOD reduction with BOD load, according to sewage strength.

gpd/cu ft, about 90 per cent efficiency of purification can be obtained. For a loading of 224 to 300 gpd/cu ft, the efficiency is 85 per cent. Even with a BOD load greater than 625 lb/day/1,000 cu ft (or about 300 population equivalent), 85 per cent efficiency is obtained in the biological treatment and when mechanical sedimentation is included, the total efficiency reaches 90 per cent.

Activated Sludge

The Detmold plant carried out biological treatment with a high suspended solids concentration in the mixed liquor. The average concentration varied from 6,000 to 12,000 mg/l. Values of this order are not usual in large-scale plants, but are possible at Detmold.

Ninety per cent efficiency was recorded for a loading of 165 gpd/cu ft when the BOD loading was 1.0 lb per lb of SS, and 85 per cent with a BOD loading of 1.2 lb per lb of SS. The sludge volume index varied between 40 and 80 and was usually about 50. Under the microscope, the sludge showed a considerable amount of life with many *Vorticellae* and *Opercularia*.

Oxygen

To avoid the possibility of having low oxygen concentrations become a limiting factor in treatment, a high rate of aeration was used during the period of experimental operation. Enough reserve oxygen was always introduced to cope with the variable quantity and strength of the sewage.

Plant Operation

Operation of the activated sludge plant was satisfactory. Primary sedimentation was good and the supply of sewage to the activated sludge plant was uniform. The steady supply of sewage over ¾ of the length of the tanks led to a high concentration of sludge in the aeration tanks. The return sludge amounted to 30 per cent of the total sewage and its SS concentration was 20,000 to 30,000 mg/l.

Summary

The activated sludge units at Detmold have operated effectively. The BOD reduction attributed to biological treatment exceeded 85 per cent. Complete biological purification of a high

strength sewage could be obtained with a retention period of little over 1 hr and a sewage load of 150 to 180 gpd/cu ft. Sewage with a BOD of about 200 mg/l can be satisfactorily transformed to one with a BOD of less than 30 mg/l with a retention period of 35 to 45 min or a load of 224 to 300 gpd/cu ft. It is, therefore, not thought necessary to add a second biological unit to the plant. Should the plywood factories start to discharge heavily contaminated wastes having an average BOD of 300 to 400 mg/l, the activated sludge plant would have to be expanded so that a maximum hydraulic load of 180 gpd/cu ft would not be exceeded. If the production of plywood were transferred to factories in other areas, as is expected from their production scheme, then the existing plant would be able to treat completely a waste with an average BOD of 200 mg/l.

CONCLUSIONS

The experiments in Kassel showed that satisfactory biological treatment of sewage could be obtained with a very short retention period in the aeration tank. Laboratory experiments carried out by Pasveer (5) were confirmed in the pilot-scale experiments in Kassel, indicating that with a retention period of 10 to 15 min in the aeration tank, a substantial BOD removal is possible. Above all, it was confirmed that in none of the experiments described was there any deterioration of purification due to excessive aeration. This fact has not been mentioned in any previously published experiments.

The experiments done by Kuisel (6) and Wuhrmann (1) in Zurich, have already shown that a BOD reduction of over 80 per cent is possible in aeration tanks with a retention period of 1 hr. These results have been confirmed in the pilot plant of Kassel and by the operation of the larger plant at Detmold. The latter plant has operated for more than two years with a retention period of 40 to 50 min. The results of the experiments and the operation of the Detmold plant show that purification can be effective, provided that certain limits of BOD load are maintained. A certain ratio must be maintained between the quantity of sludge and the BOD load.

The experiments of Pasveer (5) have been confirmed in that the oxygen supply is found to be a limiting factor for the biological purification of a normal domestic sewage. The supply must meet the demand of the microorganisms. The average dissolved oxygen content in the aeration tanks should be 2 mg/l. With a high BOD loading for the unit volume available, it is essential that there is also a correspondingly large oxygen supply. In selecting the system of aeration, it should be kept in mind that maximum turbulence should be obtained in the aeration tanks.

Acknowledgments

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MEETINGS OF INTEREST

Oct. 17–21	**48th Annual National Safety Congress,** Chicago, Ill. R. L. Forney, Secretary, National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill.
Oct. 18–21	**First Effluent and Water Treatment Convention,** The Seymour Hall, London, W. 1, England. Thunderbird Enterprises Ltd., 140 Cromwell Rd., London, S. W. 7.
Oct. 19–21	**National Society of Professional Engineers,** Statler-Hilton Hotel, Denver, Colo. National Society of Professional Engineers, 2029 K St., N. W., Washington 6, D. C.
Oct. 24–26	**Association of Analytical Chemists,** 8th Detroit Anachem Conference, McGregor Memorial Conference Center, Wayne State University, Detroit, Mich.
Oct. 25	**Association of Consulting Chemists and Chemical Engineers,** Annual Meeting, Hotel Shelburne, New York.
Oct. 29	**Conference of Wisconsin Sewage Works Operators,** Eau Claire, Wis. Roger P. Scovill, 5509 Pheasant Hill Road, Madison, Wis.
Oct. 31–Nov. 4	**American Public Health Association,** San Francisco, Calif. Berwyn F. Mattison, M.D., Executive Director, 1790 Broadway, New York 19.
Nov. 27–Dec. 2	**American Society of Mechanical Engineers,** Winter Annual Meeting, Statler Hilton Hotel, New York. L. S. Dennegar, the American Society of Mechanical Engineers, 29 West 39th St., New York 18.
Nov. 28–30	**Conference on Water Quality in the Columbia River Basin,** Washington State University, Pullman, Wash. W. H. Knight, Technical Extension Services, Washington State University, Pullman.
Dec. 4–7	**American Institute of Chemical Engineers,** Statler Hotel, Washington, D. C. F. J. Van Antwerpen, Secretary, 25 W. 45th St., New York 36.
Dec. 12–14	**National Conference on Problems and Progress in Water Pollution Control,** Sheraton-Park Hotel, Washington, D. C. U. S. Public Health Service, Washington, D. C.

SEWERAGE AND WASTE DISPOSAL PRACTICES OF SANITATION DISTRICTS OF LOS ANGELES COUNTY *

By John D. Parkhurst

Assistant Chief Engineer, County Sanitation Districts of Los Angeles County, Los Angeles, Calif.

Following World War I, Southern California experienced unpredicted and rapid population growth and industrial expansion. This was especially true in Los Angeles County where numerous, new, small communities were established and sewage treatment facilities of existing cities overtaxed by the outburst. Demand by the public was acute for all utilities, but perhaps the most perplexing problem lay in providing sewage collection and treatment facilities.

With a patchwork type of development, suitable treatment plant sites were at a premium. The state health department found that requests for permits to construct sewage treatment plants greatly exceeded available sites. Beach cities attempted to construct individual ocean outfalls, much to the consternation of neighboring cities which feared contamination of their beaches from improperly treated sewage.

Sanitation District Act

Consolidation of the many individual sewerage problems appeared to offer the best solution. In 1923, after considerable study, the County Sanitation District Act was passed by the state legislature. Under this Act, the board of supervisors of any county can organize a sanitation district with boundaries determined by geographic and topographic drainage areas and without regard to political subdivision. The primary function of such a district is to construct, operate, and maintain facilities to collect, treat, and dispose of sewage and industrial wastes. The construction of local sewers and laterals and their connection to the sanitation districts' system remained the responsibility of local authorities.

A district has broad powers which are administered by a board of directors. Since the district may contain both incorporated and unincorporated areas, each area has representation on the board; the cities, by the presiding officer of the governing body of a city, and the county area, by the chairman of the board of supervisors. By mutual agreement, the board of directors meets at least monthly and special meetings may be called if necessary. Some of the powers of the board are listed below:

1. It may employ engineering, construction, clerical, operational, and management personnel.
2. It may construct, own, and operate sewage collection and treatment works.
3. It may own and operate its works jointly with other districts or with other governmental agencies.
4. It may issue bonds.
5. It may cause taxes for district expenses to be levied.
6. It may also engage in refuse disposal.

Joint Agreements

A major reason for the success of the sanitation districts has been the

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

power to join with other districts to own, construct, maintain, and operate joint facilities. This requires only two key agreements to which a third, the Joint Refuse Disposal Agreement, was added in 1956. These two agreements are known as the Joint Administration Agreement and the Joint Outfall Agreement. All of the districts in Los Angeles County are members of the Joint Administration Agreement, while 14 have mutually owned facilities and are members of the Joint Outfall Agreement. Since 1956, nine of the districts have also participated in Joint Refuse Disposal Agreements.

Joint Administration Agreement

The Joint Administration Agreement provides for a centralized administrative organization to perform the administration of the affairs of each and all districts including construction, supervision, operation, and maintenance, and further provides that all of the districts shall have the same chief engineer, assistant chief engineers, other assistant and division engineers, secretary, accounting employees, counsel, and other personnel as may be appointed or employed.

Under the agreement, compensation of officers and employees and all other expenses of the organization are allocated or apportioned among the various districts as follows: All expenses or charges which may be charged directly to a district for whose sole benefit the expense was incurred, including salaries for design engineers, draftsmen, and others whose work may be accurately charged, are paid by the specific district. All other expenses incurred in connection with the Joint Administration Agreement which cannot be readily allocated are paid by apportionment, with each district paying its share in the ratio that its bond issue or capital investment, whichever is the greater, bears to the aggregate sum of all of the districts.

To simplify the administration of the affairs of the organization, District No. 2 in Los Angeles is designated to act on behalf of all districts on all joint matters and in effect becomes a fiscal agent for all of the districts. The Joint Administration Agreement provides for participation in the state employees' retirement system and District No. 2 is designated as the public agency for the purposes of the retirement law. The annual contribution required from each district to the retirement system is based on the ratio of salaries the individual district has paid to the sum or total salaries paid by all of the districts.

All books, records, and accounts relating to the joint administrative organization are available at all times for inspection by the board of directors of any district or to other interested parties. In the event of a dispute or controversy in connection with the books, records, or accounts, or in connection with any officers, employees, affairs, or operation of the joint administrative organization, the agreement provides for a board of arbitration. This board consists of a member of each sanitation district and the findings of the majority are binding and conclusive to all parties.

Joint Outfall Agreement

The Joint Outfall Agreement provides for joint construction, operation, and maintenance of trunk sewers, pumping plants, and treatment works to the mutual benefit of two or more districts. For the purpose of facilitating the acquisition of lands and rights of way and payment for all joint outfall capital, maintenance, and operating costs, District No. 2 is designated to act on behalf of all the districts.

The Joint Outfall Agreement specifically states the total capacity of each unit of the joint outfall system

in terms of cubic feet per second together with the percentage of ownership that each district has purchased in each installation. This agreement thereby becomes the authority for determining the amount of money to be spent by each district on joint projects as well as recording the total capacity each district owns in the entire system. Upon payment by each district of its proportionate share of the cost of the joint outfall units, perpetual right to this capacity is obtained. However, a district may purchase from or sell capacity rights in existing joint outfall units to another district provided that it is mutually desirable to do so. In this case, the cost to be paid is always considered as the original cost of such capacity. As new construction becomes necessary, each district desiring a portion of the additional capacity pays its proportion of the cost of the work according to the percentage of ownership desired.

The maintenance and operation of the joint outfall system, including upkeep and repair costs, are shared jointly by the various districts having ownership. These costs are proportioned, however, not according to the percentage of ownership in a given unit, but according to the ratio that the amount of flow contributed by an individual district bears to the total flows of all districts in that particular unit. Periodically, flow measurements are made to determine the amount of sewage originating in each district and passing through each unit of the outfall system.

Provision is made for revision of the various percentages of ownership in the individual joint outfall units from time to time as new districts are formed or as other requirements demand. It is also possible for a district to rent capacity on a temporary basis from another district. Rental rates have been established at 5 per cent of the original cost per annum.

Joint Refuse Disposal Agreements

A Joint Refuse Disposal Agreement provides that all districts so desiring may participate jointly in the construction and operation of refuse transfer and disposal facilities. District No. 2 is designated to act as the fiscal agent on behalf of each of the participating districts. For the purpose of further simplifying the entire operation, District No. 2 is also the agent for payment of all capital, maintenance, and operating expenses, including salaries, wages, or administrative expenses. The agreements recite each member district's share of the capital cost to be allocated for refuse transfer and disposal at a particular site. All operating costs, including expenses, salaries, wages, or other compensation incurred in connection with the operation and maintenance of the refuse transfer and disposal facilities are derived from a service charge for the use of the facilities. Of this unit price, a definite part is credited to a special suspense fund collected from taxes levied for financing refuse transfer and disposal. The agreements stipulate that this increment is always to be such that upon exhaustion of the disposal facilities or site, the suspense fund will have accumulated to the extent that each district may be reimbursed the amount of its total investment in acquiring and implementing the site. Provisions are also made for reinvestment of these accumulated moneys in new disposal sites and also for participation of new districts in the existing sites, should the several boards of directors so desire.

Formation of a District

Immediately following the formation of a new district, a chief engineer's report is prepared. This report indicates the area to be included, the size and location of the trunk sewers to be constructed, as well as the amount of capacity to be constructed or pur-

chased in the existing joint outfall system.

Upon approval by a two-thirds majority of the registered voters in the district, general obligation bonds are issued to finance the original construction and purchase of capacity in the existing facilities. Both the interest and sinking fund as well as the annual cost of the operation of the system are paid for by a direct tax on the real property in the district. For this purpose, the county tax collector collects taxes and places the moneys in the county treasury. Expenditure of district funds is authorized by the county auditor upon demand of the board of directors of a specific district.

Growth and Development

The first sanitation district was formed in December of 1923 and included the cities of Redondo Beach, Hermosa Beach, and Manhattan Beach as well as adjacent county territory. By 1930, nine sanitation districts had been organized in Los Angeles County. Of these, four were tributary to the city of Los Angeles treatment plant and four districts had designed their system as a joint venture. One district failed to provide financing for construction and remained inactive. All nine districts subscribed to the joint administrative organization to handle all affairs of all districts in Los Angeles County in conformance with policies instituted by the several boards of directors.

The Districts' large and well-integrated system constructed in the late 1920's performed an outstanding service during World War II.

Expansion After World War II

The success of the Districts' operation during the period of World War II laid the foundation for extensive district expansion following the war. One of the districts which had been formed in 1924, but had remained inactive, financed itself in 1945, and by 1956, 10 additional districts had been formed, voted bonds, and were in operation. Today, the combined system of 19 sanitation districts is one of the largest in the world, containing 750 miles of trunk sewers, serving 5,697 miles of lateral sewers, operating 6 treatment plants, numerous pumping plants, and disposing of 280 mgd of sewage from a 658-sq mile area containing 3 million people and 59 incorporated cities and large tracts of unincorporated county areas. All but about 16 mgd is disposed of through jointly owned district facilities.

Sewage Treatment and Effluent Disposal

The jointly owned trunk sewer system, ranging in size from 8 to 144 in. in diameter, is tributary to a large primary treatment plant known as the Joint Disposal Plant, currently treating about 264 mgd. Here, solids are removed from the sewage and industrial wastes by plain sedimentation, floatables removed by skimming, and the effluent disposed of by means of two long tunnels under the San Pedro Hills into the Pacific Ocean. The settled solids, after anaerobic digestion, are dried and sold for fertilizer. Sedimentation tank skimmings are sold to an extraction company which recovers the animal fats and vegetable oils for their commercial value.

The districts have long been an advocate of disposal of primary sewage effluent into ocean waters, the interest dating back to the early 1920's when the now-retired Chief Engineer, A M Rawn, first began investigation of the phenomena associated with ocean outfalls. To date, the districts have constructed and successfully operate three major ocean outfall pipes. One is 60 in. in diameter, extending 5,000 ft off shore and discharging into 110 ft of water; one 72 in. in diameter, extending 6,500 ft off shore and discharging into 165 ft of water; and one 90 in. in diameter, extending 8,500 ft off shore

and discharging at a depth of 215 ft. These outfalls have been designed and constructed to take advantage of favorable ocean currents, the temperature differentials between sewage effluent and ocean waters, and the great capacity of the ocean waters for waste assimilation.

While dispersion, dilution, and the thermocline, together with favorable ocean currents, generally have been sufficient, a massive chlorination system is available to maintain bacteria counts within specified limits.

In addition to the huge Joint Disposal Plant, the districts also operate and maintain five small secondary sewage treatment plants. Two of these are in the Antelope Valley north of Los Angeles where the climate is ideal for the use of oxidation ponds for secondary treatment. One plant, serving the community of Lancaster, utilizes about 230 acres of oxidation ponds to evaporate effluent from the plant treating about 2 mgd. There is no percolation to the underground in this area because of the extreme tightness of the alkali soil. The other plant, serving the community of Palmdale, treats about 0.7 mgd and sells the water from the oxidation ponds for irrigation of alfalfa.

An activated sludge plant at Pomona, constructed prior to the formation of the sanitation district in that area, has been maintained in operation, following construction of trunk sewers, as a water reclamation unit at a capacity of about 4.5 mgd. A portion of the very high quality, chlorinated effluent is sold for irrigation of pasture land and citrus groves and the remainder percolated to the underground to recharge aquifers.

Two small plants using high-rate filters for secondary treatment operate as water reclamation units. One, operating at a capacity of 0.7 mgd, treats domestic sewage from the city of Azusa. The other, owned and constructed by a large brewery, is operated by the sanitation district to treat 0.5 mgd of non-sanitary wastes from the brewery. Unchlorinated effluent from both of these plants is returned to the underground by means of percolation through coarse gravel strata.

Basic Concepts

From the first inception of the sanitation districts, the guiding philosophy has been to provide a maximum of service and to safeguard public health without undue burden to the taxpayers. To accomplish these goals, all engineering design has been conceived and carried to fruition using sound and thoroughly tested engineering principles. While an eye has been kept on simplicity of operation, automation has been thoroughly exploited to reduce personnel requirements and thereby operating costs. These principles have resulted in the full utilization of the ocean waters as an adjunct to sewage disposal, development of stage digestion of sewage solids, and utilization of sludge gas to operate gas engines. One of the more recent developments consists of construction of special hopper-bottom digestion tanks which can be cleaned while the tanks are in full operation, resulting in tremendous savings in costs and manpower. District research and development have resulted in an exceptionally satisfactory system for control of the density of the raw sludge pumping from the primary sedimentation tanks. The system, which is fully automatic, uses a radioactive source in conjunction with an ionization chamber to indicate the density of the sludge being pumped, and the pumps are in turn controlled by the density of the sludge desired. Interlocking electric controls start and stop the pumps and open and close valves; a recording instrument indicates a continuous record of the full operation. The savings in manpower and digestion tank capacity have been considerable. Dewatering of the digested sewage solids after passage

through the digestion tanks is now accomplished mechanically by means of large, continuous centrifuges. These machines, which require no chemical pretreatment of digested sludge, have a capacity of about 150 gpm each and produce a dried sludge cake which is sold for fertilizer after a nominal amount of additional drying. Centrate from the machines contains only dissolved solids along with the finer fraction of the suspended solids and can be readily disposed of through the ocean outfalls. The machines have greatly reduced the amount of odor reaching the atmosphere and at the same time they have simplified the sludge-dewatering process.

Industrial Waste Policy

Policies formed over the years by the districts have been designed to encourage industrial development. As a result, sewage reaching the Joint Disposal Plant contains a very high ratio of industrial wastes. Pretreatment of industrial wastes at the point of origin is required where such wastes may contain chemicals or solids which might be harmful to the outfall system, treatment process, or littoral waters. Surveys and studies made to date have not shown that the high ratio of industrial wastes in the primary effluent has caused any deleterious effect on the flora and fauna in the ocean. In fact, underwater photography shows that fish life abounds in the area. Industrial plants pay no surcharge for industrial wastes, but they are assessed on the basis of real property at the same tax rate as all other real property in the district.

Refuse Disposal Policy

In 1947, the Los Angeles County Air Pollution Control District ordered dump sites to cease the burning of rubbish, effecting the prompt closing of several dump sites which were not suitable for conversion to sanitary landfills and hastening the filling of remaining disposal sites. The resulting need for disposal facilities prompted the officials of several cities to request that the County Sanitation District Act be amended to include refuse disposal as one of the Districts' activities. In 1949, amendments to the health and safety code authorized the collection and disposal of refuse by Sanitation Districts; the act was later amended in 1953 and again in 1957 to limit refuse activities to those of disposal, since many cities evidenced little need for a district-wide refuse-collection system.

Basic to the refuse disposal program has been the establishment of refuse disposal facilities on a "self-liquidating" basis. Property taxes at a very modest level were assessed during the formative years of the refuse disposal program to provide a capital reserve fund for the purchase of real property, improvements thereupon, and acquisition of the necessary materials and equipment. Charges for disposal at landfills and transfer stations are established at such rates as to pay all operation and maintenance costs as well as to return to the capital reserve fund all expended moneys.

Currently, the sanitation districts of Los Angeles County are operating two major landfills and one refuse-transfer station. Two additional sanitary landfills are planned for the first quarter of 1960; the fifth sanitary landfill will begin operations during early summer 1960. Transfer stations will be provided as local private and public landfill sites are exhausted, with an ultimate program calling for 20 or more refuse-transfer stations and subsequent hauling to mountainous areas as far as 50 miles from the metropolitan area.

Organization

The Districts own and maintain a joint administrative office which is centrally located to serve the entire county. The office serves as headquar-

ters for all operations and is the meeting place for the various boards of directors. Here, under the direction of the chief engineer and general manager, all clerical and fiscal business of the districts is conducted, engineering design undertaken by the Districts' engineering staff, and plans and specifications prepared for new work. The Districts also maintain two divisional field offices, one to serve the northern portion and one to serve the southern portion of the county. From these offices, the Districts' far-flung, county-wide system of trunk sewers and pumping plants is operated and maintained by a comprehensive maintenance program. Special testing crews conduct routine tests on sewage samples from carefully selected locations throughout the system to make certain that wastes which may be harmful to the system are not discharged to the sewer. Frequent inspections are made to determine the condition of the trunk sewers and manhole structures. Flow measurements are conducted periodically with trained crews throughout the joint outfall system to properly determine the proportionate costs as required by the Joint Outfall Agreement.

These divisional offices also serve as headquarters for the field engineering staff since all new construction, except for the small amount which falls under the category of operation and maintenance, is done under contract. This staff includes well-trained construction inspectors, surveyors, and engineers. Inspection of connections to the Districts' system is expedited from these field offices which also serve as information centers on the location, size, and availability of the Districts' facilities.

Districts a Confederation

The County Sanitation Districts in reality are a large confederation of communities, joined together for mutual benefits and disposal of community wastes. Consolidation has resulted in construction of a large, well-integrated system to serve many communities which might not have been financially able to even afford sewers. Formation of the districts has in itself provided a uniform policy for all disposers throughout the county which in turn has aided industrial and residential development. This has produced great savings, both in construction and in operation. The organization has made available to each community, through membership in a sanitation district, a well-trained staff of engineers and scientists and through their efforts guaranteed the protection of public health by means of efficient waste disposal facilities.

Summary

Experience has shown that formation of a sanitation district in a new area materially increases property values, simplifies sewage disposal problems, and encourages the general development of the area. The unqualified success of the sanitation districts in Los Angeles County has already prompted formation of numerous similar districts in other counties in the state of California.

Activities and operations of the County Sanitation Districts have been directed by the late A. K. Warren and, until recently, by A M Rawn. At present, C. R. Compton serves as Chief Engineer and General Manager.

Industrial Wastes

TREATMENT OF COMBINED SEWAGE AND FRUIT CANNING WASTES *

By John T. Norgaard, Ronald Hicks, and Donald A. Reinsch

Civil Engineer, Brown and Caldwell, San Francisco, Calif.; Chief Chemist, Auckland Metropolitan Drainage Board, Auckland, N. Z.; and Civil Engineer, Brown and Caldwell, San Francisco, Calif.

Disposal of sewage and industrial wastes from San Jose, California, and vicinity to the tidewaters of southeast San Francisco Bay has been a problem for at least 30 years. There is a high concentration of fruit canning industries in the area, and the waste assimilative capacity of that portion of the bay is limited. These factors cause seasonal gross pollution of the waters and result in hydrogen sulfide production and a widespread odor nuisance.

Following the recommendations of a 1946 county-wide survey and report (1), plans were completed in 1950 for primary sedimentation and sludge digestion, with secondary treatment in oxidation ponds to be situated in tidal marshland adjacent to the plant. Litigation over land for the ponds delayed the program for five years and resulted in virtual abandonment of plans for ponds as a sole means of secondary treatment. As a first step in corrective action, a 36-mgd primary treatment plant was placed in service in December 1956 (2). Due, however, to the nature of the fruit waste, no measurable improvement in receiving water quality was achieved during the 1957 canning season. In May 1958, the city authorized a one-year study of and report on (*a*) receiving water capacity, (*b*) applicable secondary treatment processes, and (*c*) preliminary designs and costs of feasible alternative plans for treatment and disposal. In April 1959, approval was obtained for a seven-month extension of this undertaking. This paper deals with the second phase of the work, namely, the experimental studies of applicable treatment processes.

Wastewater Sources

The San Jose sewage treatment plant, in which Santa Clara recently acquired an interest, serves those two cities, five other municipalities, and six sewerage districts—practically all the urbanized parts of the Santa Clara Valley. The location of San Jose is shown in Figure 1.

The population of metropolitan San Jose is growing rapidly. In 1950, there were about 120,000 persons in the potential sewerage service area of 300 sq miles. Now there are 400,000 persons, about 350,000 of them tributary to the system. Projections indicate a tributary population of 700,000 by 1970, and 1,200,000 by about 1985 (Figure 2).

Supporting this population are intensive valley farming and an industrial complex ranging from the historic prune-drying operations to those involved in nuclear power and space

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

FIGURE 1.—Location map of San Francisco Bay area and San Jose.

flight. Wastes from rapidly-growing nonseasonal industries exert a total BOD equal to that from the contributory population. The seasonal canning industry, although stable in size, remains a major source of manufacturing employment (Figure 3) and a principal contributor to the waste disposal problem.

National production figures indicate the size of the San Jose-Santa Clara canning industry: 23 canning plants served by the wastewater treatment plant process 12 per cent of the peaches, 20 per cent of the pears, and 45 per cent of the fruit cocktail canned annually in the United States. Extending generally from late July to mid-September, the canning season is marked by a four-week peak during which more than 5,000 tons of fruit are processed daily, six days per week. The peach-pear season is preceded by a small apricot pack, and is overlapped and followed by a substantial tomato pack.

All the canning plants are scattered within the central areas of San Jose

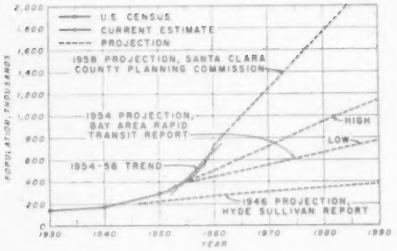


FIGURE 2.—Santa Clara County population.

and Santa Clara, both of which are about seven miles from the treatment plant. Obviously, therefore, separate collection systems for the seasonal wastes would not be feasible. It would, however, be feasible to obtain partial separation of (*a*) the growing suburban areas and (*b*) the more stable central sections, including the canning operations. This plan is considered in developing the treatment alternatives.

Combined Waste Characteristics

As shown in Figures 4, 5, and 6, plant loading in terms of volume, suspended solids, and BOD rises sharply during the four-week fruit processing peak. In the 1958 peak, the average weekday flow rose from 32 to 55 mg, suspended solids loading from 130,000 to 220,000 lb, and BOD loading from 100,000 to 420,000 lb.

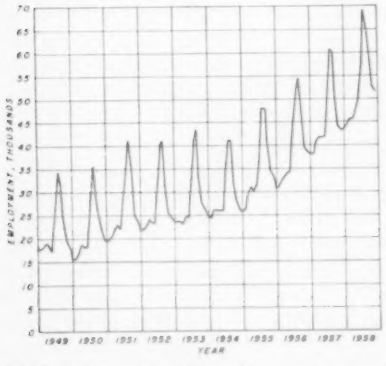


FIGURE 3.—Manufacturing employment in Santa Clara County.

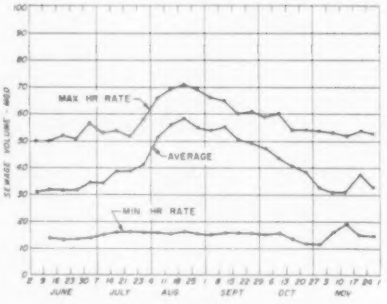


FIGURE 4.—Raw wastewater flow to San Jose plant in 1958.

In terms of composition, the combined sewage and waste is marked by a high BOD (Table I), which averaged 920 mg/l in August 1958, and high total volatile solids—1,410 mg/l during the same month. As a result of the use of fine screening by the canners, suspended solids were 480 mg/l, or slightly lower than in the off-season months. Suspended matter, however, was flocculent and formed a light voluminous sludge. Settleable solids by volume averaged 35 ml/l in August 1958, compared to 10 ml/l in a typical off-season month.

It is evident that most of the BOD of the combined sewage and fruit waste is exerted by dissolved organic matter, primarily fruit sugars and acids. As a result, removal of BOD by primary treatment, normally 35 to 40 per cent in off-season, drops to 20 to 25 per cent during the processing peak.

Influent pH values in the 5.0- to 6.0-range are attributable to fermentation

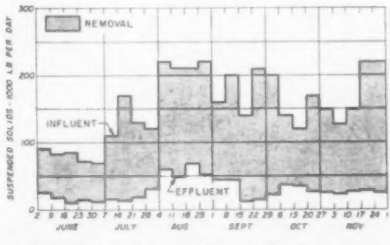


FIGURE 5.—Suspended solids data at the San Jose plant in 1958.

of sugars and their conversion to organic acids and carbon dioxide during the four-hour transit to the treatment plant. Fermentation is aided by the high average temperature of 28.7°C. Total organic nitrogen concentration of 34 mg/l indicates a BOD : N ratio of 25, which is considered nearly optimum for biological treatment. In reality, a gross deficiency in readily available nitrogen exists since the BOD : NH_3-N ratio is in the range of 150 to 200. It is suspected that ammonia nitrogen is utilized by fermentation organisms during the four-hour transit to the plant and that the high organic nitrogen content may be predominantly due to cell material of live organisms rather than waste proteins. Phosphate concentrations, on the other hand, are about 20 mg/l and are ample for biological treatment.

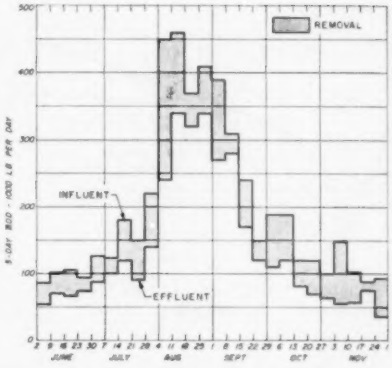


FIGURE 6.—BOD data at the San Jose plant in 1958.

Another important characteristic in selecting and designing a suitable method of treatment is an extremely high rate of oxygen uptake by the sew-

TABLE I.—Sewage Characteristics, Week Days, August 1958 [a]

Determination	Plant Influent			Plant Effluent		
	Average	Maximum	Minimum	Average	Maximum	Minimum
5-day 20°C BOD	920	1,460	600	680	870	530
Suspended solids[b]						
Total	480	960	260	120	250	50
Volatile	360	660	160	—	—	—
Settleable solids (ml/l)	35.3	53.5	14.0	8.7	24.0	4.9
Total solids	2,480	3,270	1,950	1,930	2,190	1,570
Total volatile solids	1,410	1,870	1,050	1,080	1,440	600
pH	6.0	6.6	5.5	5.6	6.5	4.9
Alkalinity	270	360	180	220	340	110
Chloride	136	298	90	—	—	—
Total sulfide	1.0	2.5	0.4	1.1	1.8	0.4
COD (3 min)[c]	53	68	32	40	51	32
COD (4 hr)[c]	320	410	260	240	280	180
Ammonia nitrogen[d]	5.1	8.5	1.7	4.3	8.5	0
Organic nitrogen[d]	32.4	34.5	30.1	20.3	22.3	18.4
Total nitrogen[d]	36.4	39.3	31.8	22.4	22.9	21.9
BOD/NH_3-N	180	—	—	160	—	—
BOD/total N	25	—	—	30	—	—
Oxygen utilization rate (mg/l/hr)	20[e]	—	—	—	—	—
BOD rate constant, k, at 20°C	0.38[f]	—	—	0.40[f]	—	—
Temperature (°C)[g]	28.7	30	27	—	—	—

[a] Except as noted, all analyses results are of weekday composites and are expressed as mg/l.
[b] By filter paper method. Effluent sample results are lower than obtained with standard asbestos mat.
[c] Cold permanganate method. Analyses made only August 20 to 31.
[d] Four analyses only of samples collected August 21, 25, 26, and 27.
[e] Average value for 4 hr at 29.3°C.
[f] August 7 sample. At temperature of measurement, 29.3°C, k = 0.54 and 0.58.
[g] August 21 to October 1.

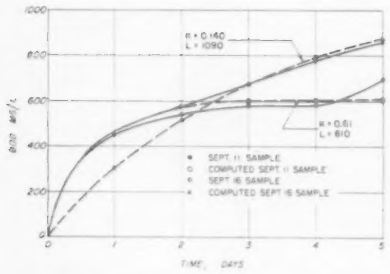


FIGURE 7.—Effect of time on influent BOD.

age. For raw sewage, 1-day BOD results were 65 per cent of the 5-day; for settled sewage, 73 per cent. At 28.5°C, oxygen uptake by settled sewage, determined by the Warburg manometric respirometer procedure, ranged from 35 to 45 mg/l/hr over an 8-hr period. Assuming the monomolecular equation is applicable to the BOD of this waste, the reaction rate constant k computed from the two 5-day series (Figure 7) is equal to 0.615 for the first 2 to 3 days. The curve for the sample taken September 16, practically at the close of the fruit canning season, appears to have a second carbonaceous stage with a k value of 0.14. Nitrification accounted for only 40 mg/l of the 5-day demand. In summary, the 5-day BOD of the combined sewage and waste of San Jose is at best a crude measure of its oxygen demand and is not comparable to the 5-day demand of a normal sewage. Caution must be taken, therefore, in comparing or applying normal design factors and performance criteria to the San Jose problem.

Anticipated growth of population and nonseasonal industry, with no change in the fruit processing, would result in the design loadings indicated in Figure 8. For the next decade, sewage and waste volume during the peak month is expected to rise from 55 to 94 mgd, while BOD is expected to increase but 25 per cent, from 420,000 to 535,000 lb/day. The resulting change in concentration and other characteristics must be taken into consideration in process selection and design.

Experimental Program

Three general methods of treatment were investigated:

1. Anaerobic primary plus aerobic secondary treatment by activated sludge and high-rate trickling filter.
2. Primary sedimentation plus secondary treatment by high-rate filter, both single-stage and two-stage.
3. Activated sludge process, including several modifications thereof, both with and without primary sedimentation.

Different degrees of treatment were investigated for all of the general processes. Such information is needed to compare costs of alternative projects involving longer outfalls and lower degrees of treatment versus shorter outfalls and higher degrees of treatment. The 1958 bay studies showed the desired degree of treatment to range from 76- to 92-per cent BOD removal. In addition, the use of oxidation ponds following an intermediate degree of treatment is being considered.

Both bench-scale and pilot treatment facilities were operated, the latter be-

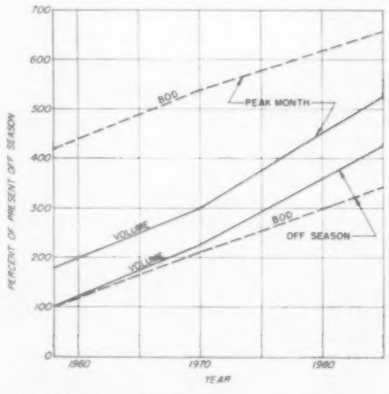


FIGURE 8.—Projected San Jose volume and BOD.

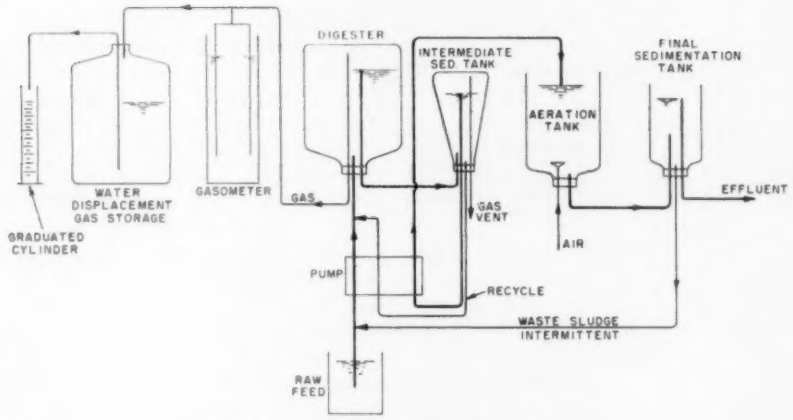


FIGURE 9.—Profile of 1958 laboratory units.

ing studied only during the 6- to 8-week fruit canning season.

In general, laboratory procedures followed "Standard Methods" (3). Exceptions included the suspended solids determination. In 1958, the filter paper method was used, which gave low results for settled and final effluents, even with one-liter samples. In 1959, the glass-fiber filter method outlined by Nusbaum (4) was used, and gave entirely satisfactory results. Volatile acids were determined in 1959 by the ether extraction method of Thomas *et al.* (5).

The bench-scale unit (Figures 9 and 10) was set up in the treatment plant laboratory. It included a digester for anaerobic treatment of the whole waste, gas collection and measuring facilities, an intermediate sedimentation vessel, a second-stage aeration vessel, and a final settling tank. Feed was continuously pumped to the digester by a pulsating pump that also served to recycle first-stage sludge to the digester,

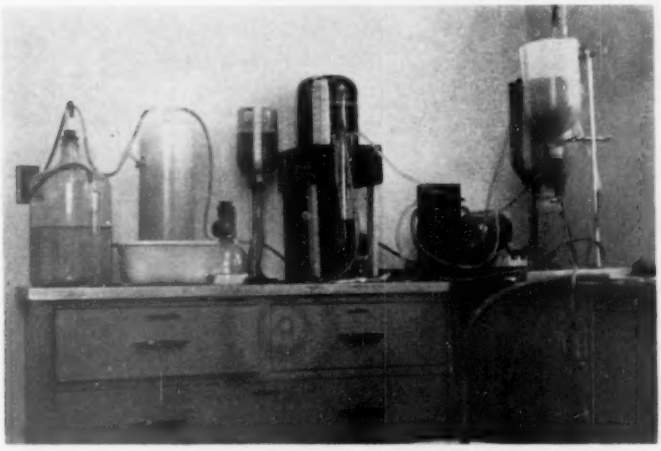


FIGURE 10.—Laboratory units.

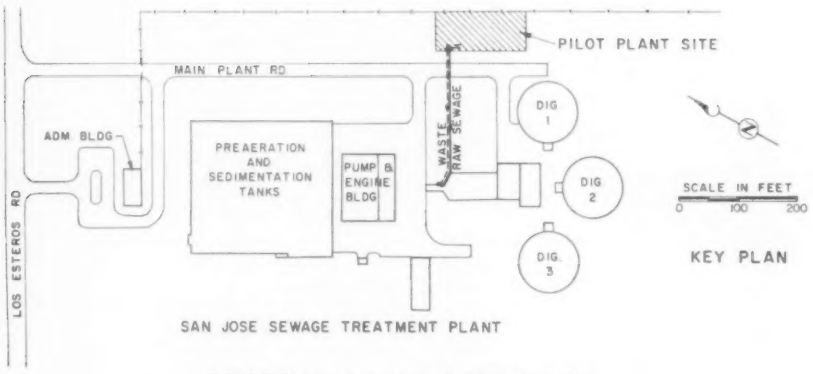


FIGURE 11.—Location of pilot plant site.

and as desired, second-stage sludge to the aeration tank. Excess activated sludge was fed to the digester manually, usually once a day. The whole operation, including digestion and storage of feed and effluents, was at room temperature—generally 80 to 85°F.

The pilot plants were built near the pretreatment building of the main plant. Sewage was pumped from the Parshall flume, which is downstream from the bar screens and grit channels (Figure 11). At the pilot plants (Figure 12), the influent passed through a small comminutor to a distribution channel 20 ft above ground. All the plants were served by a single- and 3-phase power supply, low-pressure air, high-pressure digester gas supplies, and

FIGURE 12.—Pilot plant for experimental studies of treatment of sewage and cannery wastes. Layout provided for simultaneous operation of nine independent treatment plants.

a sump pump and piping for returning effluents and waste sludges to the main plant influent channel. A field laboratory provided for control tests and refrigerated storage of composite samples.

Principal pilot plant units consisted of 39 tanks, made of stacked concrete pipe, and 9 trickling filters. These comprised three parallel plants or batteries for each of the three types of processes (Figure 13). Digestion and aeration tanks were in multiple units of 8-, 4-, 2-, and 1-hr detention, at a nominal flow of 40 cfh (5 gpm), to provide necessary flexibility. These tanks ranged from 66 to 27 in. in

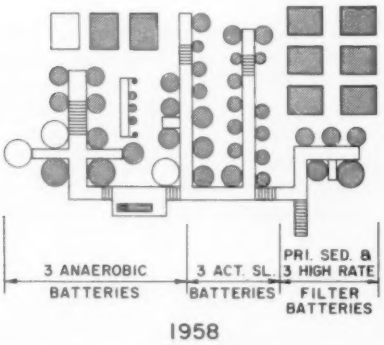


FIGURE 13.—1958 pilot plant layout and tanks in service.

diameter and had water depths of 12 to 14 ft (Figure 14). For digester stirring, gas from the main plant was injected near the bottom. The digesters were left open in 1958 but were equipped with floating covers in 1959.

Each aeration tank was equipped with a sparger in 1958. Because it was found that the small, deep tanks provided insufficient vertical velocity for sparger action, plastic-wrapped diffusers were used in 1959 in conjunction with an air blower installed as a pressure booster. Also in 1959 two aeration tanks were equipped with variable-speed turbine diffusers having ring spargers.

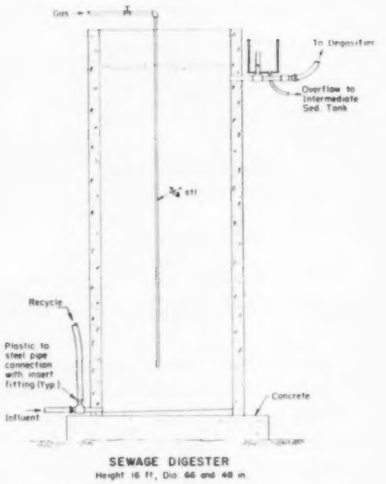


FIGURE 14.—Profile of digester.

All sedimentation tanks except the large primary tank had a diameter of 42 in., a side water depth of either 5 or 7 ft, and a 60° cone bottom. Total water depths were 8 and 10 ft (Figure 15). Low inlet velocities at high rates of recycle were achieved by using an annular skirt for circumferential distribution. At the design flow rate of 40 cfh, detentions were 1½ and 2 hr for the 8- and 10-ft depths. The surface

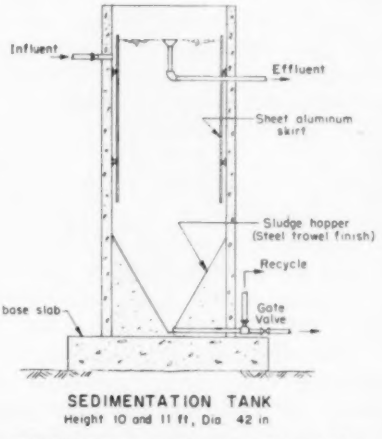


FIGURE 15.—Profile of sedimentation tank.

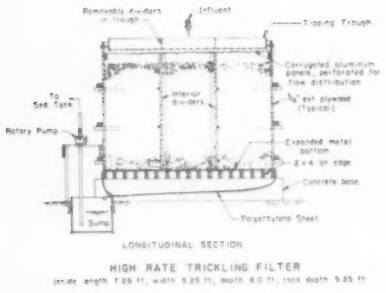


FIGURE 16.—Profile of trickling filter.

rate was 4.6 fph, corresponding to 830 gpd/sq ft, and the effluent weir rate was 8,000 gpd/ft.

Each of the trickling filters (Figure 16) contained 193 cu ft of 3- to 6-in. rock having a depth of 5 ft, with a surface area of 38.6 sq ft. Influent was distributed by a tipping trough and a perforated, corrugated aluminum tray above the rock. Filter effluent, after passing through a wooden grill rock support, was collected on a polyethylene sheet and discharged to a receiving sump. Both the tipping trough and filter had three sections so that loading rates could be altered.

Each anaerobic battery had a vacuum degasifier, containing 32 trays, mounted between pole supports (Figure 17). Rotary vacuum pumps were controlled by mercury manometers to hold the vacuum within ¼ in. of the 21-in. level.

Pumps for recycling sludges and filter effluents were a rotary positive displacement type with neoprene impellers. Variable speed drives provided a range of 12 to 200 cfh. Air lifts were also used in recycling activated sludge, and splitter boxes were used to proportion waste and return flows and to enable measurement of recycle rates. Erecting and altering plant piping was made much easier by the use of flexible polyethylene pipe, but insert fittings tended to aggravate troubles due to air binding.

Anaerobic Primary Plus Aerobic Secondary Units

The anaerobic process tested was essentially identical with that developed for meat wastes (6) (7), except that the digesters were not heated. In that process, the whole waste is passed into a tank of actively-digesting sludge having a concentration in the mixture of 10,000 to 20,000 mg/l. Sludge solids in the digester effluent are then separated by sedimentation and returned to the digester. To get consistent solids separation, degasification of the digester effluent by exposure to vacuum or by other means is usually necessary.

Laboratory Unit

The laboratory unit was operated almost continuously from May to mid-October 1958, but was not set up in 1959. Operation was divided into 10 periods or runs, each marked by changes in loading, feed characteristics, or method of operation.

Before and after the fruit canning

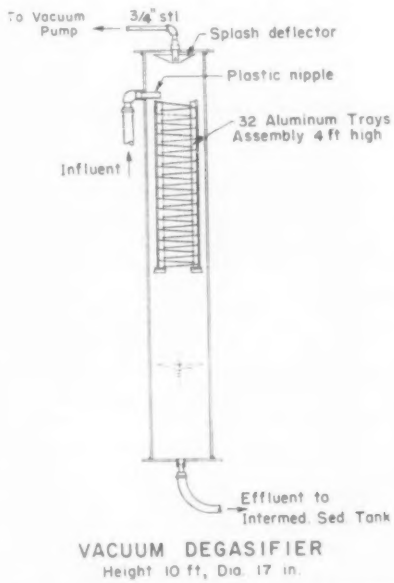


FIGURE 17.—Profile of vacuum degasifier.

TABLE II.—Summary of Laboratory Plant Loading and Performance

Item	Series								
	2	3	4	5	6	7a	7b*	9	10
Test period	6/9 –28	6/29 –7/4	7/5 –18	7/18 –24	7/24 –8/8	8/8 –25	8/25 –28	9/4 –17	9/17 –10/11
Elapsed time (days)	19	6	13	6	16	17	4	13	24
Anaerobic stage									
Digester detention (hr)	6.6	4.2	4.6	4.5	4.2	4.4	4.0	4.4	4.2
5-day 20°C BOD									
Influent (mg/l)	960†	1,200†	840†	620†	770†	780‡	310*	540‡	870†
Effluent (mg/l)	340	580	340	210	230	315	141	270	440
Loading (lb/1,000 cu ft)	330	430	270	210	270	260	110	180	310
Efficiency (per cent)	64	52	59	66	70	60	55	51	49
Gas produced									
(Cu ft per lb BOD applied)	2.1	1.8	3.0	3.0	1.8	3.3		2.8	2.8
Carbon dioxide (per cent)	—	12.0	7.8	6.0	6.8	6.2		5.4	8.0
Methane (per cent)	—	75.5	81.3	83.2	83.2	82.5		84.3	80.0
Aerobic stage									
Aerator detention (hr)	5.9	3.8	4.1	3.9	3.7	3.8	3.4	3.9	3.6
5-day 20°C BOD									
Influent (mg/l)	340	580	340	210	230	200§	141	270	440
Effluent (mg/l)	87	150	53	52	91	32§	8.7	34	50
Loading (lb/1,000 cu ft)	86	230	125	80	93	79§	61	110	190
Efficiency (per cent)	75	75	85	76	61	84§	94	87	89
Total BOD removal (per cent)	91	88	94	92	88	96§	97	90	93

* Raw sewage from city of Stockton South Plant, August 25–28 inclusive, average BOD, 300 mg/l.

† Raw San Jose sewage fortified with homogenized canned dietetic peaches and pears.

‡ Raw San Jose sewage, not fortified.

§ Seven-day period, August 18–25 only.

season, feed consisted of raw sewage fortified with homogenized dietetic canned peaches and pears. The fortified sewage was held at room temperature overnight, and gently agitated with compressed air to allow fermentation to begin and thus to simulate the condition of the combined wastewater on arrival at the treatment plant. The prepared batch was then sampled for analysis and fed to the digester over a period of 24 to 48 hr.

It was unnecessary to stir the digester contents. About every 15 min gas trapped in the sludge developed enough buoyancy to lift the entire mass to the top of the vessel. The gas was freed in the accompanying turbulence and the sludge quickly resettled.

In the nine periods reported (Table II), digester detention, based on average feed rate, was 6.6 hr for the first run and 4.0 to 4.6 hr for the rest. On a volumetric basis (Figure 18), BOD loadings ranged from 180 to 430 lb/1,000 cu ft of digester capacity for seven of the periods, and 110 lb for an unexpectedly weak sewage for the 8th. First-stage or primary BOD removal ranged from 49 to 70 per cent. Gas production ranged from 1.8 to 3.3 cu ft/lb of BOD applied, and methane content ranged from 75 to 84 per cent.

Solids separation without degasifica-

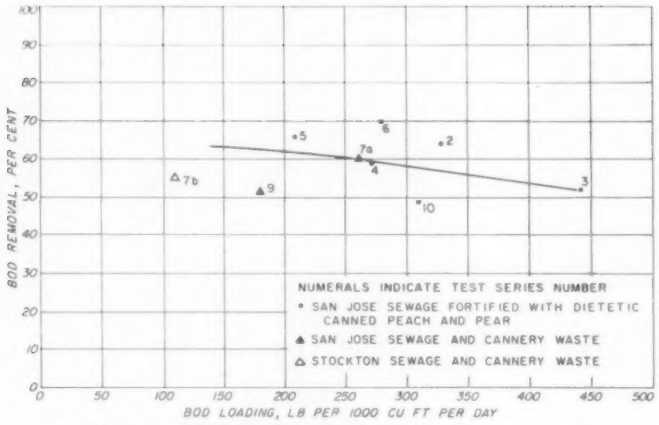

BOD REMOVAL, PER CENT
BOD LOADING, LB PER 1000 CU FT PER DAY
NUMERALS INDICATE TEST SERIES NUMBER
• SAN JOSE SEWAGE FORTIFIED WITH DIETETIC CANNED PEACH AND PEAR
▲ SAN JOSE SEWAGE AND CANNERY WASTE
△ STOCKTON SEWAGE AND CANNERY WASTE

FIGURE 18.—1958 laboratory treatment efficiency in terms of BOD removal.

tion was never a problem. The SDI was normally in the range of 2.9 to 4.0; the sludge was granular in structure, and settled and compacted rapidly.

The anaerobic effluent was low in suspended solids, but high in total sulfide. With a pH normally in the range of 7.1 to 7.6, the odor was predominantly fishy or amine-like.

In the second stage, the aeration period (Table II) was 5.9 hr during the first series and from 3.4 to 4.1 hr in the other 7. After the third period, the return of activated sludge from the final sedimentation tank to aeration was discontinued. Without adequate means for wasting sludge on an approximately continuous basis, overnight sludge buildup was excessive. In subsequent runs, activated sludge growth was adequate without any return. Large growths of *Sphaerotilus natans* and of *Beggiota alba*, however, tended to accumulate in the aeration vessel because of the relatively small size of the glass tubing outlet. This situation was controlled by periodic cleaning of the aeration unit.

BOD loadings of the aerobic second stage varied from 60 to 230 lb/1,000 cu ft of aerator volume. BOD efficiency ranged from 60 to 94 per cent, and, apparently because of the variable activated sludge concentration, it exhibited little if any relation to the foregoing loading parameter. Over-all removal for both stages was between 88 and 97 per cent. Highest removal was with unfortified waste during the peak of the fruit canning season.

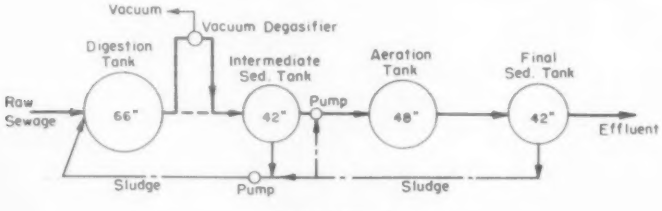


FIGURE 19.—Flow diagram of anaerobic primary and aerobic secondary for Battery I, 1958.

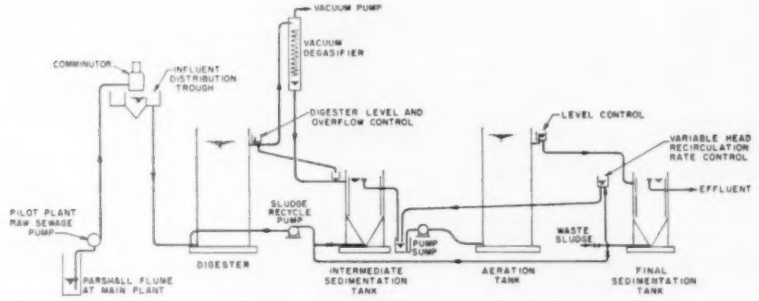


FIGURE 20.—Schematic flow diagram of Battery I, 1958.

The final effluent was generally clear, and odor-free, and the effluent collected over 24 hr normally contained dissolved oxygen. The average 5-day BOD ranged from 32 to 91 mg/l for 6 of the 8 series. It was 9 mg/l for the lowest digester loading and 150 mg/l for the highest.

1958 Anaerobic Pilot Plant Units

Although 24-, 16-, and 8-hr digester detention was possible in the three anaerobic plants or batteries, the successful operation of the laboratory unit at much lower detention led to the adoption of 8-, 5.3-, and 4-hr detentions. The digesters were seeded with fine-screened sludge from the main plant digesters, and after three to four days of low feed input, were placed "on stream." One battery was provided with an aeration tank and final settling tank for secondary treatment (Figures 19 and 20), and the other two with high-rate filters and final tanks.

Active digestion, as shown by BOD removal and production of significant quantities of methane, failed to develop during the 1958 pilot plant operation. Results at no time approached those attained in the laboratory plant. Alkalinity and pH values would increase for a few days, but on the verge of reaching optimum levels for methane fermentation, they invariably fell. Changes in feed rates, and such operational procedures as recycling, gas stirring, and degasifying, resulted in better performance in certain respects but failed to bring about any marked improvement in digestion. In fact, the best results were attained during the first five days of operation. As a consequence of the low BOD removals, secondary units were overloaded and performance was poor.

However, based on cost estimates derived from the laboratory unit, the City of San Jose approved a second season of pilot plant investigation.

1959 Anaerobic Pilot Units

Digesters for four batteries, one of which utilized former activated sludge tanks (Figure 21), were seeded with active sludge in late June 1959, and placed under low feed by July 1. The

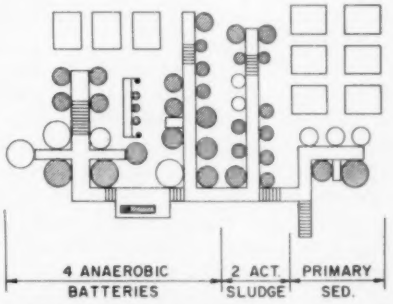


FIGURE 21.—1959 pilot plant layout and tanks in service.

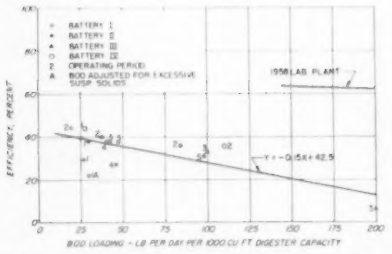


FIGURE 22.—Degree of BOD removal in anaerobic primary, 1959.

apricot pack, producing some fruit waste during July, was expected to minimize the shock loading of the peach-pear pack by gradual acclimatization of the sludge. Floating covers were installed on the digesters and three of the four batteries were provided with activated sludge second-stage facilities.

By July 15 it was found that a continuing drop in pH was caused by the high carbon dioxide content of the gas used for digester stirring. In a full scale plant the stirring gas would be recycled and thus be stripped only once of its carbon dioxide. The pilot plant, however, used a continuous stream of fresh "foreign" gas. This condition was overcome by installing a pressurized scrubbing tower and using a soda ash solution to reduce the carbon dioxide content to 10 per cent. Installation of equipment needed for adequate control of the scrubber operation was not completed until early August, and several short periods of high carbon dioxide caused the occasional drop in pH in late July.

Despite the correction of the gas supply, methane production failed to pick up with the advent of the strong fruit waste in August, and BOD removal continued low. Moreover, even at a very low feed rate, pH and alkalinity dropped in all digesters, and volatile acids rose from the 200- to 300-mg/l range to 500 to 700 mg/l. Analysis of the influent for volatile acids showed weekday values in the range of 300 to 350 mg/l. Finally, it was noted that the ammonia nitrogen in the digester effluents had dropped from a range of 30 to 50 mg/l to about 15 mg/l, comparable to that of the raw sewage, indicating a decline in the liquefaction stage of digestion.

As a correction, the pH of the feed to the fourth battery, beginning on August 8, and to the first two batteries on August 15, was adjusted to maintain pH values of 6.8 or higher in the digesters. Anhydrous ammonia was used for that purpose, as a matter of convenience, and not because of possible supplemental nutrition. The pH of the feed to the third battery was not adjusted. In that battery, the pH of the digester dropped to 5.5, the volatile acids rose to 800 mg/l, and on August 26 the battery was taken out of service.

For the rest of the season, with digester detentions of 9 to 21 hr, alkalinity and pH conditions were favorable and total volatile solids in the sludge increased only slightly. While gas production improved, it did not approach the values reached in the 1958 laboratory unit. Volatile acids in the digester closely followed the values found in the feed.

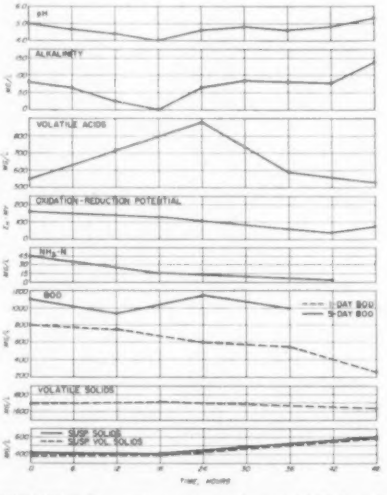


FIGURE 23.—Characteristics of 1959 batch feed experiment.

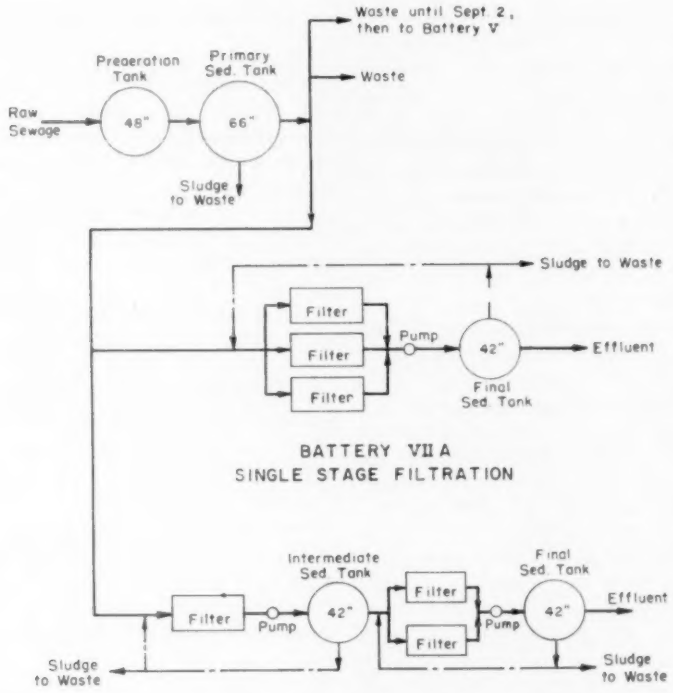


FIGURE 24.—High-rate filter batteries, 1958.

BOD loadings and efficiency for selected periods of operation in 1959 (Figure 22) were far lower than in the 1958 laboratory plant, and much of the time BOD removal was little greater than that obtained by primary sedimentation. As compared with the pilot plant results in 1958, however, digestion improved. Furthermore, BOD removal by secondary aeration and settling was generally good, although loadings were lower than originally planned.

A laboratory study of sludge acclimated to 35°C was made in one-liter flasks equipped for gas collection and measurement. After gas production had stabilized, 520 mg/l of glacial acetic acid was fed for three days at approximately 14-hr intervals. Gas production increased, but was far short of theoretical values. At the end of the run, 500 ml of supernatant was replaced with an equal quantity of raw sewage and fruit waste. On the second day thereafter, the volatile acid content of the sludge was 370 to 400 mg/l, or still in excess of that in the sewage feed. In a parallel operation at 24°C, the sludge failed to utilize the acetic acid.

Other studies planned in connection with the anaerobic process, including those pertaining to sludge removal rates in the sedimentation tanks and to aeration versus vacuum treatment for degasification, were not undertaken in view of the failure to obtain an acceptable rate of digestion. An examination was made, however, of the effects of 48-hr storage of feed at room temperature, as practiced in the opera-

tion of the laboratory unit in 1958. Results showed marked changes in characteristics, even though the BOD remained substantially the same (Figure 23). These data indicate that much dissolved volatile matter was converted to suspended volatile solids during the 48 hr. Results varied in two replications of the test, each using a batch with quite different initial characteristics but in general confirmed the above conclusion.

It appears, therefore, that in the laboratory unit the mechanics of BOD removal may have primarily involved: (*a*) adsorption of suspended and colloidal matter, including a dispersed bacterial growth, developed during storage and (*b*) the subsequent methane fermentation of the adsorbed matter. The fruit waste as received, however, requires a process involving direct and rapid methane fermentation of dissolved carbohydrates, including volatile acids. The necessary environment and flora to attain that objective were not developed in these pilot plant investigations.

High-Rate Filters

High-rate filters were used as a secondary treatment process only during the 1958 canning season. Because of the ultimate 150-mgd size of the treatment works, filters would require an excessively large area. As in most large metropolitan works providing secondary treatment, the activated sludge process was found more economical. For this reason, a minimum of data is presented here on the high-rate filter operation and performance.

On July 24, 1958, the filters were placed under low load at high recirculation and a week later were further seeded with secondary sludge from a nearby filter plant. The units were operated in single stage for an additional week, then one battery was provided with three single-stage filters and a second battery with one first-stage filter, two second-stage filters, and an intermediate as well as a final sedimentation tank (Figure 24). Despite the foregoing seeding period, an additional two weeks of operation under de-

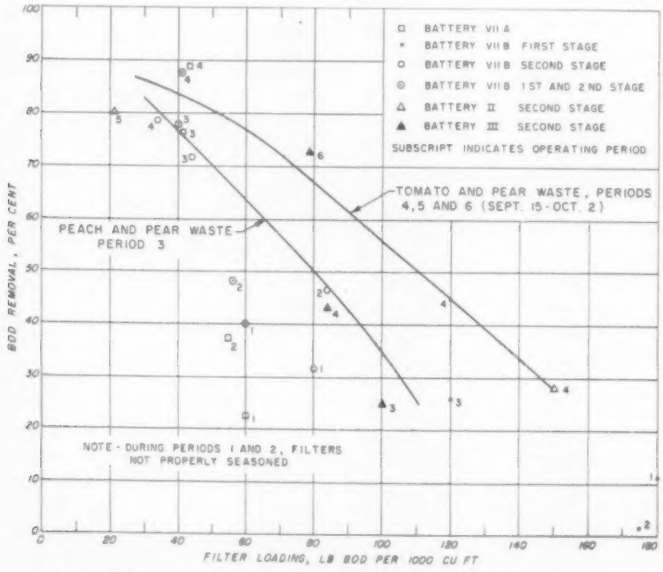
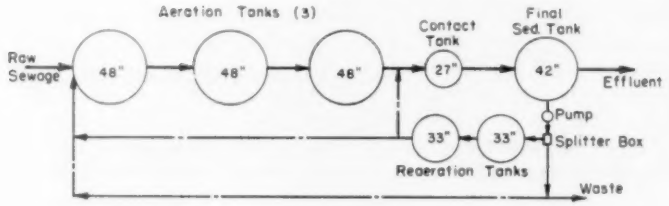


FIGURE 25.—Performance of high-rate trickling filter, 1958.



BATTERY IV

FIGURE 26.—1958 activated sludge plant.

sign load was required to mature the filters.

Loading and performance data on pounds of BOD applied per 1,000 cu ft (Figure 25) indicate two distinct phases of performance: (*a*) during the balance of the peach-pear canning season, August 22 to September 15, and (*b*) during the tomato pack and dwindling pear pack, from September 15 to October 2. Performance during the latter period was superior to the first.

Activated Sludge

In 1958 the following flow patterns were used in testing a variety of activated sludge and other aeration schemes:

1. Plain aeration of raw sewage and final sedimentation without return of sludge.
2. Plain aeration of raw sewage, followed by adsorption on activated sludge, final sedimentation, and reaeration of the return sludge.
3. Primary sedimentation, followed by the pattern outlined in No. 2 above.
4. Activated sludge treatment of raw sewage without sludge reaeration.
5. Procedure 4 with occasional use of reaerated sludge to maintain a stable sludge.

In 1959 the operational plan was primary sedimentation followed by activated sludge treatment utilizing sludge reaeration.

In addition to operating the pilot plant units, investigations of oxygen uptake by the sewage and activated sludges were made both years, using the Warburg manometric procedure. At the close of the 1959 season, oxygenation capacity of the several types of aeration devices were determined for the various tank sizes and shapes. Dye tests were also made of sedimentation tank detention characteristics.

1958 Activated Sludge Pilot Plants

The three activated sludge plants (Batteries IV, V, and VI) provided total aeration times of 15, 11, and 9 hr at the design rate of 40 cfh (5 gpm). Operation began August 11 at half the design feed rate, and the rates of flow as well as the operational procedures varied during the subsequent 8 weeks of tests. A generalized flow diagram is shown in Figure 26.

Data on BOD loadings and removals (Figure 27), as in the case of the high-rate filters, show significantly poorer results during the peach-pear pack, compared to the subsequent tomato-pear canning period. Of the several methods of operation, normal activated sludge treatment gave the highest BOD removals at given loadings.

Spot checks for dissolved oxygen showed frequent deficiencies in aeration capacity in the initial stage of aeration. It was also difficult to maintain dissolved oxygen in sludge reaeration tanks when raw sewage was fed to a plant.

About September 1, the problem of sludge bulking arose in the two batteries using normal activated sludge treatment. The condition was alleviated, but not entirely corrected, by adding reaerated sludge to the system.

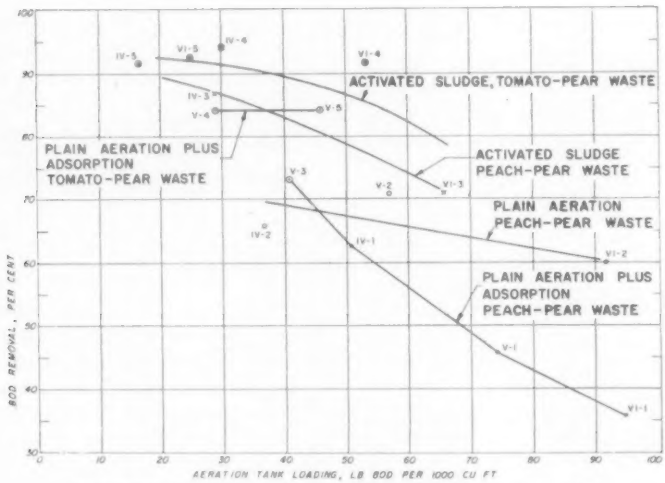

ACTIVATED SLUDGE, TOMATO-PEAR WASTE
ACTIVATED SLUDGE PEACH-PEAR WASTE
PLAIN AERATION PEACH-PEAR WASTE
PLAIN AERATION PLUS ADSORPTION TOMATO-PEAR WASTE
PLAIN AERATION PLUS ADSORPTION PEACH-PEAR WASTE
BOD REMOVAL, PER CENT
AERATION TANK LOADING, LB BOD PER 1000 CU FT

FIGURE 27.—Performance of activated sludge unit, 1958.

Bulking did not occur in Battery V, using plain aeration of settled sewage followed by adsorption and sludge reaeration.

Except in the first stages of plain aeration, intensive frothing of the aeration tanks occurred in both 1958 and 1959.

1959 Activated Sludge Pilot Plants

Batteries V and VI, providing total aeration times of 11 and 9 hr at the design flow of 40 cfh, were used for activated sludge in 1959. Flow patterns (Figure 28) for both plants allowed a variable portion of the aeration capacity to be utilized for sludge reaeration, and the balance for mixed liquor aeration. Step aeration also was used for several periods in Battery V. As mentioned, the air supply was increased over that of 1958. Beginning August 8, 35 mg/l of nitrogen in the form of anhydrous ammonia was added to the settled sewage feed from noon to 4 AM each week day.

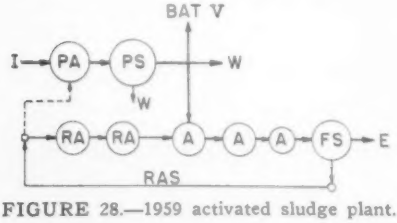


FIGURE 28.—1959 activated sludge plant.

Six periods of operation were studied for each battery. These differed in BOD loading, detention, or activated sludge solids carried in the system. The first two, in July, tested the feasibility of an intermediate degree of treatment of the weaker, off-season sewage, using relatively low sludge concentrations. In the next four periods, during the fruit processing season, detention periods were gradually reduced from 13.6 to 3.8 hr, based on average feed rates. In comparison, aeration time in 1958 ranged from 27 to 10 hr. Table III summarizes 1959 data.

For the 6 periods with an average feed rate of 60 cfh, the surface rate in the final sedimentation tanks was 7.2 fph or 1,300 gpd/sq ft. At this high rate, carry-over of activated sludge solids occurred on a number of days. To differentiate the aeration phase and solids separation phase of the process, BOD determinations were also made of laboratory-settled effluent samples and are indicated as adjusted values.

Performance during the canning season is shown in three ways. Figure 29

shows volumetric loading in terms of total aeration detention versus efficiency. The spread of points at detention periods of less than 8 hr indicates that other factors, principally sludge solids in the system, affect the performance. In this, as well as in later figures, data for Periods 1 and 2, prior to the canning season, are plotted but not considered in the regression curves.

The second measure of loading, pounds of BOD applied per 1,000 cu ft of tank volume (Figure 30), is of interest primarily as an indicator of construction cost. Although a linear regression curve has been fitted to all points, two curves better satisfy actual conditions of operation. Comparable loading curves for 1958, without supplemental nitrogen feed, appear on the left.

The third measure of loading, pounds of BOD applied per 100 lb of activated sludge solids (Figure 31), is the best parameter of the three. As observed elsewhere, a break in performance oc-

TABLE III.—Summary of Loading and Performance, Activated Sludge, 1959

Item	Battery V				Battery VI			
Operating period	3	4	5	6	3	4	5	6
(Dates)	8/3	8/11	8/19	8/31	8/3	8/11	8/19	8/30
	–11	–19	–30	–9/17	–11	–19	–30	–9/17
(Days)	8	8	11	13*	8	8	11	13*
Nominal detention (hr)†								
Aeration	7.0	7.2	4.8	2.8	3.3	3.3	2.2	2.3
Reaeration	6.2	6.4	4.2	2.9	6.3	6.5	2.9	1.5
Activated sludge solids (mg/l)								
In aeration tanks	1,240	1,380	1,480	950	1,260	1,130	1,460	860
In reaeration tanks	2,060	4,080	4,260	2,780	2,850	3,010	4,590	2,400
Sludge volume index (SVI)	470	270	220	670	250	240	170	640
Returned aerated sludge (per cent of influent volume)	50	34	25	35	39	31	22	35
5-day 20°C BOD								
Loading factors								
(lb applied per 1,000 cu ft aeration)	59	64	87‡	132	81	89	153‡	200
(lb applied per 100 lb activated sludge solids)	57	39	50‡	113	58	60	78	220
In effluent (mg/l)	42	31	41‡	90‡	110	45	46‡	160‡
Efficiency (per cent)								
Primary treatment	18	23	18	23	18	23	18	23
Secondary treatment	92.0	94.7	92.2‡	82‡	79	92.4	91.4	68‡
Over-all	93.4	95.9	93.6‡	86‡	83	94.2	92.9‡	75‡
Suspended solids								
Efficiency (per cent)								
Primary treatment	57	65	36	73	57	65	36	73
Secondary treatment	69	77	73‡	–220	75	70	72‡	–210
Over-all	87	90.5	83‡	14	89	88	82‡	13
Excess activated sludge (lb per lb BOD removed)	0.28	0.50	0.71	1.00	0.49	0.61	0.87	0.80

* Excluding Sept. 6, 7, 13, and 14.

† Based on influent volume; in addition: preaeration 1.0 hr, primary sedimentation 1.9 hr, final sedimentation 2.0 hr in Periods 3 and 4 and 1.3 hr in Periods 5 and 6.

‡ Data for secondary units in Periods 5 and 6 adjusted to allow for carry-over of excess activated sludge returned to primary.

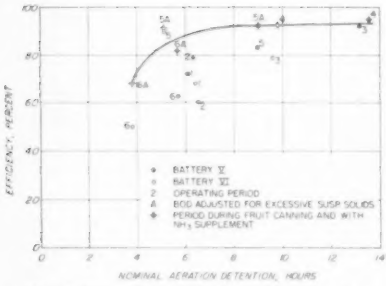


FIGURE 29.—1959 activated sludge efficiency in terms of detention.

curs in the region of 90-per cent efficiency. At higher efficiencies, the data are reasonably fitted by a straight line, while below 90 per cent a hyperbolic curve applies. For the latter, the curve has been drawn to parallel the trend of data from 30 activated sludge plants as reported by Haseltine (8). The significant shift of the curve to the right reflects the characteristics of the waste, including temperature.

Controlling the desired sludge concentration levels was sometimes difficult, as in the period between August 18 and 29, when recycling of waste activated sludge through the primary sedimentation tank inadvertently occurred in four successive days. During August 10 through 29, with supplemental ammonia feed, sludge volume indexes were stable and within workable limits. During August 30 to September 17 bulking occurred in both batteries with the development of *Sphaerotilus natans* and a consequent heavy loss of sludge in the effluent. When no

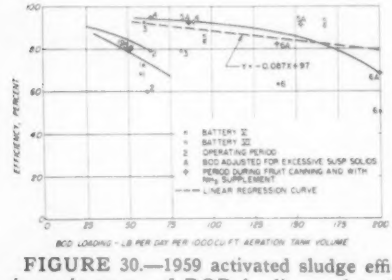

EFFICIENCY, PERCENT
BOD LOADING - LB PER DAY PER 1000 CU FT AERATION TANK VOLUME
Y = -0.087X + 97
BATTERY Σ
BATTERY ΣΣ
OPERATING PERIOD
BOD ADJUSTED FOR EXCESSIVE SUSP SOLIDS
PERIOD DURING FRUIT CANNING AND WITH NH3 SUPPLEMENT
LINEAR REGRESSION CURVE

FIGURE 30.—1959 activated sludge efficiency in terms of BOD loading and tank volume.

recovery occurred during the Labor Day week end, the sludge was dumped and a new growth started with reaerated sludge on hand from the prior period. This sludge suffered the same fate. Chlorination of return sludge, started September 10, was entirely effective in Battery V and partly effective in Battery VI. On the whole, sludge concentrations August 29–September 17 were far below design levels.

In Battery VI during the last two weeks, aeration detention solids loading (220 lb BOD per 100 lb of sludge), was apparently at or beyond the line of demarcation for stable operation. Loadings for Battery V, however, were initially lower than during the stable operation of Battery VI in the preceding operating period. Since bulking also began at approximately the same date in 1958, the concurrent increase in tomato wastes may have been at least partly responsible for the bulking condition.

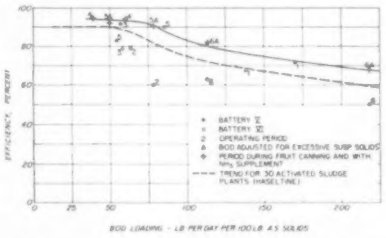

EFFICIENCY, PERCENT
BOD LOADING - LB PER DAY PER 100 LB. A.S. SOLIDS
BATTERY Σ
BATTERY ΣΣ
OPERATING PERIOD
BOD ADJUSTED FOR EXCESSIVE SUSP SOLIDS
PERIOD DURING FRUIT CANNING AND WITH NH3 SUPPLEMENT
TREND FOR 30 ACTIVATED SLUDGE PLANTS (HASELTINE)

FIGURE 31.—1959 activated sludge efficiency in terms of BOD loading and aerator solids.

Aeration Requirements

Basic data for estimating aeration requirements were obtained by relating oxygen uptake rates (Figure 32) for mixed liquor and reaerating sludge, as determined in the Warburg apparatus, with operating and performance conditions for the specific days of the Warburg tests. The average uptake rate for time equal to average aerator detention (based on feed plus return flow) is herein assumed to equal the uptake rate in an aeration tank with

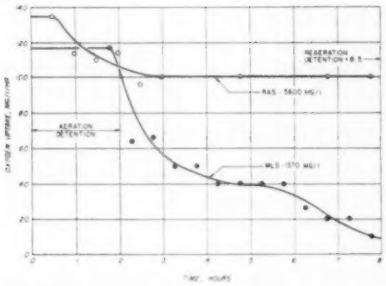


FIGURE 32.—Oxygen uptake rate versus time in Battery IV, September 16, 1959.

continuous feed. This assumption gives results on the high or conservative side because of quite complete mixing in the tank. In a more detailed analysis of data, the estimate may be refined by taking into account the normal concentration-time distribution curve in a well-mixed tank.

Oxygen uptake is expressed in terms of milligrams per liter per hour; in the example shown, it averages 112 mg/l/hr for the 2-hr detention time in the aeration tank, and 107 mg/l/hr for the 8.5-hr detention of return activated sludge in reaeration. These are extremely high uptake rates. For convenience in general application, uptake may be expressed as milligrams per gram of activated sludge solids per hour. On this basis, all measured sludge reaeration uptakes, for example, fell between 10 and 20 mg/g/hr.

Oxygen uptake of activated sludge solids is a function of the BOD applied to the solids. This relationship is indicated by Figure 33. Since the data are for individual days of operation rather than for an extended period,

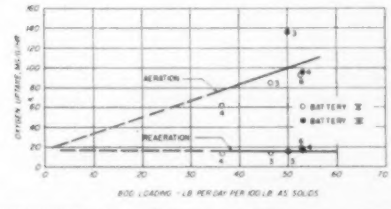


FIGURE 33.—Oxygen uptake rate versus BOD per activated sludge solids loading.

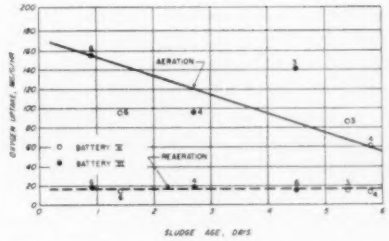


FIGURE 34.—Oxygen uptake rate versus sludge age.

the curve for mixed liquor aeration can only be considered as a line of trend.

Oxygen uptake by the sludge is also related to sludge age. This inverse relationship is shown in Figure 34. Again the curve indicates the trend. Sludge age was determined as the weight of solids in the system divided by the weight of solids wasted per day from the system.

Providing aeration capacity to meet the indicated oxygen demands will be a major problem in design of a full-scale plant.

Excess Activated Sludge

A second problem in plant design will be estimating the amount of excess activated sludge during the canning season and providing waste sludge treatment and disposal facilities.

Oxidation of sludge solids, particularly in reaeration, decreases the amount of excess sludge as sludge age

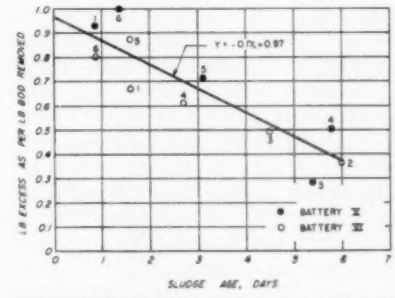


FIGURE 35.—Excess activated sludge solids per BOD removed versus sludge age.

increases. This inverse relationship is shown in Figure 35 in terms of pounds of excess activated sludge solids per pound of BOD removed, versus sludge age. The linear regression curve for the plotted data indicates values ranging from 0.37 lb of excess sludge per lb of BOD removed at a sludge age of 6 days, to 0.87 lb at 1 day.

Summary and Conclusions

Three basic processes applicable to the treatment of a large volume of combined sewage and fruit canning waste at San Jose, California, were investigated during two canning seasons on a pilot-plant scale: (*a*) anaerobic primary treatment followed by either activated sludge or high-rate filter second-stage, (*b*) primary sedimentation plus secondary treatment by both single and two-stage high-rate filters, and (*c*) activated sludge, both with and without primary sedimentation.

The anaerobic fermentation process failed to remove sufficient BOD to warrant further consideration in this project, although it showed promise in bench-scale operation.

Because high-rate filters would not be physically suitable for the large treatment works that eventually will be required for the San Jose metropolitan area, they were not studied beyond the first season of the investigation.

The activated sludge process, incorporating primary sedimentation, sludge reaeration, and supplemental feed of ammonia during the peak month of the peach-pear season, gave a high degree of treatment with relatively low aeration detention times. From the standpoint of flexibility and areal requirements, it is best suited to the conditions encountered at San Jose. Data from the investigation will serve as a basis for preliminary designs and cost estimates for alternative plans of treatment and disposal. These will differ in degree of treatment and length of outfall, and will include consideration of oxidation ponds for tertiary treatment.

Acknowledgments

Appreciation is expressed for the assistance given by Harold J. Flannery, City Engineer of San Jose; and Frank M. Belick, Engineer-Superintendent of Sewage Treatment, and his staff. George J. Schroepfer, Professor of Sanitary Engineering, University of Minnesota, acted as a consultant in 1959. Several items of equipment were loaned by Chicago Pump Company, Link-Belt Company, and Infilco Inc.

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RAPID MODIFICATION OF THE PYRIDINE–BENZIDINE METHOD FOR CYANIDE

By Myron Brin *

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For studies on the conversion of thiocyanate to cyanide by mammalian blood (1) it was necessary to have a rapid semi-micromethod for cyanide which was suitable for routine application to large numbers of samples.

After applying methods involving Prussian blue, pyridine-pyrazolone, and pyridine-benzidine, Nusbaum and Skupeko (2) reported that the latter appeared the most suitable for these purposes. Their procedure was essen-

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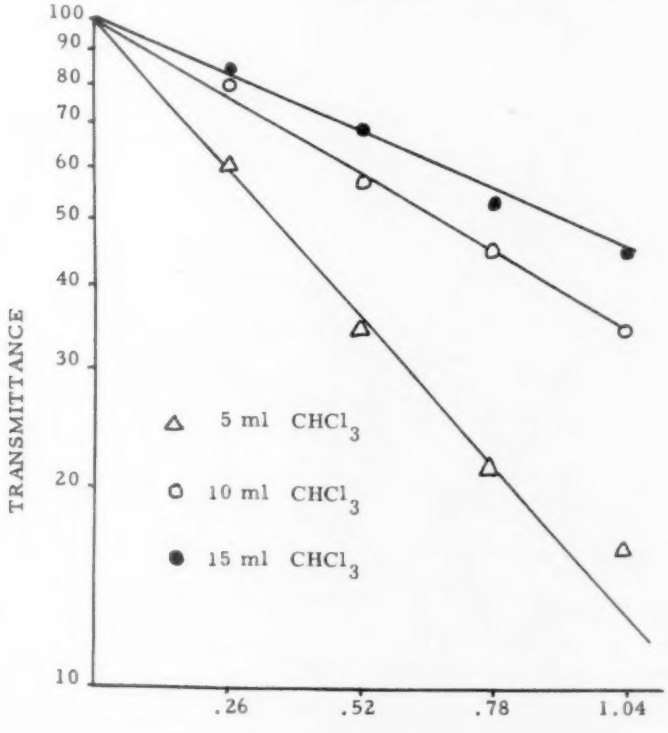

FIGURE 1.—Calibration of cyanide using modified pyridine-benzidine method.

tially as follows: The sample was acidified with phosphoric acid, and the cyanide was converted to the cyanogen bromide by the addition of bromine water, after which the excess bromine was removed with sodium arsenite. The cyanogen bromide was extracted with n-butyl alcohol with shaking, and the pyridine-benzidine reagent was added to the sample-butyl alcohol, followed by additional shaking. The color was allowed to develop for 15 min, the n-butyl alcohol layer (supernatant) was removed to the cuvette, and the color density was measured in a spectrophotometer at 480 mμ.

The most time-consuming and expensive step in the procedure as applied to processing samples was the need to transfer the n-butanol supernatant to a quartz cuvette for the final colorimetric analysis. It was believed that by using a solvent which was heavier than water, the need for transfer might be eliminated. A number of solvents were tried, and it appeared that chloroform was the most suitable. Accordingly, standard 18 × 150 mm lipless test tubes were calibrated to constant light transmission with the aid of colored solutions. A standard curve of reference was prepared with amounts of cyanide ion ranging from 0 to 1.04 μg. The reaction was carried out as described, and before reading the tubes in a spectrophotometer at 480 mμ, the tubes were tapped lightly to permit the escape of the aqueous phase from the subnatant chloroform extract. The tubes were then wiped and read directly in the colorimeter.

It was of further interest to determine whether the method could be made more or less "micro" by the use of different volumes of chloroform. The family of curves which resulted from the use of 5, 10, and 15 ml of solvent are presented in Figure 1. The use of a heavier-than-water extraction solvent, plus the ability to vary the volume of solvent as needed, and the avoidance of expensive cuvettes did much to expedite the program.

Acknowledgments

Acknowledgment is expressed to Mr. Abbott Miller and Misses Elsie Puro and Deborah Smith for technical assistance.

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Stream Pollution

ASSIMILATION OF ABS BY AN ACTIVATED SLUDGE TREATMENT PLANT–WATERWAY SYSTEM

By Emanuel Hurwitz, R. E. Beaudoin, T. Lothian, and M. Sniegowski

Director of Laboratories, Assistant Sanitary Chemist, and Junior Sanitary Chemists of The Metropolitan Sanitary District of Greater Chicago, Chicago, Ill.

There has been a considerable amount of study to determine the effect of synthetic detergents on the various phases of sewage treatment and on the receiving stream. In this country, Sawyer and his associates at the Massachusetts Institute of Technology have been the chief investigators. A summary of their work was published in 1958 (1). Most of this work and the work of other investigators have been confined to laboratory experiments designed to simulate treatment plant and stream conditions, and except in the case of "frothing" studies, little has been reported on actual plant or stream experience.

The Metropolitan Sanitary District of Greater Chicago is in a unique position to obtain data on the assimilation of ABS in a full-scale treatment works–waterway system. The District collects the sewage of some 4,800,000 people plus an industrial load having a population equivalent of approximately 3,500,000, treats it by the activated sludge process in its three major sewage treatment works, and discharges the effluents through its system of waterways at a common point in the system. It also maintains a continuous survey of the Illinois Waterway (2) with sampling stations appropriately located along its course, and it has access to reliable flow data both from its own measurements and from measurements supervised by the U. S. Geological Survey.

Sampling

Figure 1 shows the location of the three major treatment works and ten waterway sampling stations. Effluents from the treatment works are conveyed through the system of streams and canals to a point one mile downstream from Lockport where it is discharged into the Des Plaines and Illinois River system (2).

Samples from Lockport, Joliet, Marseilles, Peoria, and Pekin were composites of 24 hourly aliquots. The others were composites of samples taken hourly over 8- or 16-hr periods.

Flow Measurements

Flow data were available from the records of The Metropolitan Sanitary District of Greater Chicago and from the U. S. Geological Survey. The volume of raw sewage, primary effluent, and final effluent discharged from the Southwest Treatment Works is a firm figure taken from the Sanitary District records. The Summit flows are the sum of the flow from the Southwest Treatment Works plus the average flow from the North Side Works plus the average diversion of lake water through the Wilmette Pumping Station and through the locks at the mouth of the Chicago River. Lockport

FIGURE 1.—Location of three major treatment plants and ten sampling stations.

flows are firm figures from the records of The Metropolitan Sanitary District of Greater Chicago averaged for the days on which samples were taken. The Joliet flows are the sum of the flow at Lockport plus the flow from the Des Plaines River. Smiths Bridge flows are identical with Joliet since there are no tributaries of any consequence discharging to the system between these stations. Morris flows include the flow past Joliet plus the flows from the Kankakee and DuPage Rivers. The flow at Marseilles is a firm value taken from the records of the U. S. Geological Survey. The flow past Peoria is the flow measured at Kingston Mines by the U. S. Geological Survey minus the flow from the Mackinaw River. Since the flow measured at Kingston Mines also includes the flows from the Fox and Vermillion Rivers, it is assumed that the flow past Henry and Lacon will be approximately the same as that past Peoria.

The times of flow between stations are reasonable approximations based on the average flow past the station for the period of the test.

The weights of ABS at the various stations are based on the average of the flows on the days for which samples were available.

Analytical Procedure

ABS was determined by the method of Degens, Van Der Zee, and Kommer (3) modified to obtain more complete extract of the ABS from the solid fraction of the waste as suggested by Foote (4). The method complexes the anionic synthetic detergent with methylene blue. The complex is extracted with

chloroform and the optical density of the chloroform solution is determined with a spectrophotometer at 600 mμ in a 1-cm cell.

Two sets of samples were submitted to the Robert A. Taft Laboratories of the U. S. Public Health Service for a check on the reliability of the method. The samples were put through the cleanup procedure used for infra-red determinations and the ABS determined by methylene blue.

The results obtained by the Taft Laboratory and The Metropolitan Sanitary District of Greater Chicago Laboratory are shown in Table I. The Taft Laboratory was satisfied that the results are "quite consistent and certainly indicate that the interferences are not too numerous." The reference standard used by the Sanitary District was assayed by the supplier to contain 51 per cent ABS; a sample analyzed at the Taft Laboratory indicated an ABS content of 48 per cent. The use of this lower value would tend to narrow the discrepancy between the Taft Laboratory and the Sanitary District laboratory on the samples before cleanup. Nevertheless, the differences are well within the limits of error of the method, and the data reported are a reliable indication of the amounts of ABS at the various stations.

Assimilation of ABS by the Southwest Treatment Works

McGauhey and Klein (5) studied the ability of sewage treatment processes to remove or destroy detergents. They report that the normal reduction resulting from 2 hours' settling time was from 2 to 4 per cent. Sedimentation, therefore, is not an effective process for removing ABS. They also report that in prolonged aeration in an activated sludge unit during which the BOD removal was between 88 and 97 per cent, ABS removals varied from 19 to 77 per cent. Greater ABS reductions were achieved with higher mixed-liquor solids. At 5,000 to 6,000 mg/l mixed-liquor solids, ABS removal was 60 to 70 per cent in a two-stage unit. A single-stage unit under 4 to 6 hr aeration consistently removed 50 to 60 per cent of the ABS.

Table II shows the averages of the results obtained at the Southwest Treatment Works. The detention period of the primary settling tanks at this plant is approximately 30 min. During this short period, only the large and denser particles are removed. Since any removal in the primary tanks would be by adsorption on the surface of the particles, it is reasonable to assume that little or no ABS would be removed by the large discrete particles settled

TABLE I.—Check Analysis for ABS by Taft Laboratory and Metropolitan Sanitary District

Sampling Station	ABS (mg/l)					
	Analyses Reported 8/6/59			Analyses Reported 8/25/59		
	Taft		MSD	Taft		MSD
	Cleanup	Before Cleanup	Before Cleanup	Cleanup	Before Cleanup	Before Cleanup
Raw sewage	3.15	4.3	3.46	2.75	3.50	3.72
Settled sewage	1.36	2.3	2.80	2.37	2.97	3.40
Final effluent	0.83	1.3	1.64	0.59	1.12	1.32
Lockport	0.90	0.97	1.12	0.58	0.82	0.88
Smiths Bridge	0.50	0.87	1.10	0.76	0.48	0.94
Marseilles	0.80	0.76	0.70	—	—	—
Peoria	0.44	0.39	0.52	0.56	0.35	0.30
Pekin	0.45	0.35	0.54	0.55	0.66	0.30

TABLE II. Assimilation of ABS at the West–Southwest Sewage Treatment Works

Sample	Flow (cfs)	ABS		
		(mg/l)	Lb/day	Reduction (%)
Raw sewage	475	3.41	2.71×10^4	0.0
Preliminary effluent	468	3.49	2.76×10^4	0.0
Final effluent	468	1.60	1.265×10^4	54

in the tank. The data indicate that none was removed. However, these data should be interpreted with respect to the limitations of the analytical method.

The data also indicate a 54-per cent removal of ABS by the aeration system and confirm the observation of McGauhey and Klein that "some 50 per cent of the ABS in raw sewage may be removed by an ordinary sewage treatment plant having activated sludge as the secondary treatment process."

Assimilation of ABS in the Illinois Waterway

It has been predicted that future water shortages in many parts of the country will be so severe that some surface water will be reused as many as six times and that the discharge of wastes from the manufacture of exotic chemicals and the increasing use of synthetic detergents will seriously affect the use of these surface waters (6). It is only within the last year or so that any water from the Illinois Waterway has been used for domestic supply. Now Peoria draws part of its supply from the river. Until that time its use was confined to cooling purposes, boiler makeup, etc. But even in this area of water use, the effect of detergents was evident, particularly in boiler water makeup, where the detergent buildup in the boiler of an industry located on the waterway was roughly proportional to the makeup water and caused serious foaming problems.

It has been shown by the investigations of Sawyer and his associates (1), McGauhey and Klein (5), and by the work reported above that effluents from plants using trickling filters or activated sludge as secondary treatment will carry significant amounts of detergents to the receiving stream.

It is of interest to note the assimilation of ABS in a stream like the Illinois River which as yet has not been affected by reuse. The waterway, however, has characteristics which are not common to all streams. One of these is a series of pools along its course which were formed by the construction of dams and locks built to make it a deep waterway. These pools certainly must be a significant factor in the removal of ABS and other organic pollutants by enhancing the self-purification potential of the stream biologically, by extending the time element, and physically, by providing extensive settling basins wherein the more minute particulate matter can be removed.

It has already been noted that biological degradation of ABS proceeds at a slow rate. Renn and Barada (7) have shown that although adsorption on fine natural silts and settling has only limited value in removing ABS, there is a decrease of 8 to 15 per cent by this device. Generally, conditions in the Illinois Waterway are very favorable for assimilation of ABS.

Table III describes the process of assimilation in the Illinois Waterway. Approximately 12,600 lb/day of ABS are discharged with the effluent from the West–Southwest Treatment Works. This amount is increased at Summit, which includes the flow from the North Side Works, to 17,000 lb. At Lockport, there is a further increase which includes the discharge through the Sag Channel of the highly industrialized Calumet area. The weight of ABS continued to increase because of addi-

tions from the tributaries (Des Plaines, Kankakee, and DuPage Rivers) and reached its peak at Morris. Up to this station, gains because of additions outpaced the slow removals by biological degradation, physical adsorption, and settling. From Morris to Peoria, about 100 miles, there are no significant contributions of ABS. There is, therefore, a slow, steady assimilation of ABS by the stream. In this reach of the waterway, 7,800 lb/day are removed, approximately 27 per cent.

Peoria and Pekin are cities with a combined population of 137,000. If the pattern of ABS assimilation were consistent with that found in the upper reaches of the waterway, an increase of ABS in the stream at Pekin would be indicated. However, a decrease is noted. This suggests possible removal by forces other than biological or physical. Chemical reaction between the detergent and a waste peculiar to that area may have effected some removal.

Summary and Conclusions

These studies confirm the laboratory and pilot-plant tests of others that detergents of the alkyl aryl sulfonate type are partially degraded by the activated sludge process. Approximately 50 per cent of the ABS was removed in the treatment works. The other half was discharged with the plant effluent to the waterway. In the upper 53 miles of the waterway, the quantity of ABS in the stream increased because of additions from communities situated on the waterway and its major tributaries. The slow processes of biological degradation and physical adsorption and settling were not able to keep up with the new additions. For the next hundred miles, further additions of ABS were insignificant and assimilation by the stream was gradual and continuous. The assimilation continued past Peoria and Pekin, a distance of 39 miles, despite the additions of ABS by these communities. This continued assimilation suggests the influence of a third force, possibly chemical, in removing ABS from the stream.

But from the peak load at Morris through the last station at Pekin, a distance of 114 miles or approximately 8 days' travel time, only 37 per cent of the ABS was removed.

The resistance of ABS and other wastes from processing new and exotic chemicals forecasts grave danger to surface-water supplies. It adds emphasis to the warning by Hollis and McCallum (6) that new parameters for measuring pollution must be established and bold and heroic measures to discover new and better ways to handle

TABLE III.—Assimilation of ABS in the Illinois Waterway

Station	Distance from Southwest Works (miles)	Distance from Preceding Station (miles)	Approx. Time between Stations (days)	Avg Flow (cfs)	ABS		
					(mg/l)	(lb ÷ 10^6)	Gain or Loss (lb ÷ 10^6)
Final effluent (Southwest Works)	0	0	—	1,467	1.595	1.26	—
Summit	2.8	2.8	0.6	2,500	1.260	1.71	+0.45
Lockport	25.3	22.5	2.2	3,895	1.110	2.33	+0.62
Joliet	30.0	4.7	0.5	4,303	1.090	2.53	+0.20
Smiths Bridge	38.1	8.1	0.9	4,303	1.065	2.47	−0.06
Morris	52.6	14.5	1.2	5,927	0.899	2.88	+0.41
Marseilles	69.0	16.4	1.3	6,262	0.802	2.71	−0.17
Henry	120.0	51.0	3.5	6,945	0.588	2.21	−0.50
Lacon	127.0	7.0	0.5	6,945	0.559	2.10	−0.11
Peoria	153.5	26.5	1.8	6,945	0.518	1.94	−0.16
Pekin	166.0	12.5	0.8	6,945	0.486	1.82	−0.12

the changing character of our present-day municipal and industrial wastes must be taken.

Acknowledgments

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GREAT LAKES BASIN—ILLINOIS WATERWAY STUDY

The Public Health Service has started a six-year anti-pollution study of the United States jurisdiction of the Great Lakes Basin—Illinois waterway. The study is designed to aid development of a comprehensive plan to control and prevent pollution in the area.

Under a recent Congressional $500,000 appropriation which will cover the first year of the project work is being started for intensive studies of the kinds and amounts of municipal and industrial wastes being discharged into the waterways within the metropolitan area of Chicago, and the pollutional effects of these wastes in the waterways in Chicago.

A 40-man staff of engineers, chemists, biologists, and other scientists and technicians is being assembled. Project headquarters will be in Chicago. William Q. Kehr is Project Director.

A Special Master of the Supreme Court is now conducting hearings in cases which concern the use of Lake Michigan water and to which the States of Wisconsin, Michigan, Illinois, Minnesota, New York, Pennsylvania, and Ohio and the United States are parties. The data gathered in the first phases will assist the Court in making its decisions.

A CONSTANT FLOW APPARATUS FOR TOXICITY EXPERIMENTS ON FISH *

By Florian Grenier

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The present trend in toxicity studies is toward investigating long-term effects of toxic substances on fish on exposure to sublethal concentrations. This has particular importance in pollution control when quality criteria of water must be defined for which fish will continue to prosper in good condition rather than merely survive.

In recent years, chlorinated hydrocarbons have been used as fish toxicants in lake management. It has been found that, owing to the high stability of these compounds, treated waters will detoxify only very slowly and may remain toxic for many years. It is therefore important to ascertain, before restocking, the minimum residual toxicity that can be tolerated by fish.

Answers to such questions can be obtained in the laboratory only by experiments of long duration. For obvious reasons, control of environmental conditions in the aquarium can be obtained more easily with a constant flow of toxic solution. Renewal of the solution at a sufficient rate will correct for any alteration caused by the metabolic activity of the fish.

Whenever a high rate of replacement is not needed, continuous renewal of the toxic solution from a large batch is satisfactory. In some cases, it is more practical to use some kind of proportioning device in which a concentrated toxic solution is mixed in constant proportion with dilution water. Types of constant flow apparatus have been described by Herbert and Merkens (1) and Brebion (2).

Description of Apparatus

The apparatus used in this laboratory is shown diagrammatically in Figure 1. It consists essentially of a siphon bottle *A* which fills up and empties periodically, and a Mariotte bottle *B* which delivers a constant volume of the toxic solution at each cycle.

Siphon bottle *A* is fed with the dilution water from a constant head reservoir *R* by means of a glass tube *C*, the diameter of which is chosen to obtain the desired rate of flow. Funnel *D* is an ordinary powder funnel; the inside diameter of the stem must be large enough to allow free escape of air. The end of the stem does not extend beyond the rubber stopper, so that the water will run along the wall of the bottle, thus keeping the air content close to saturation. This is desirable especially when cold air-saturated dilution water must be heated many degrees, since superaerated water may be detrimental to fish. The small cup *E* inserted over the siphon tube *S* at the bottom of bottle *A* insures a sharp break of the siphon at the end of each cycle. The cup is preferably made from a piece of large bore glass tubing fitted with a rubber stopper which will absorb the shocks in the vertical movement of the cup, thus eliminating any risk of breakage.

At each cycle, a fixed volume of the concentrated solution or suspension is sucked into the siphon bottle. The stock solution is contained in a 4.5-l

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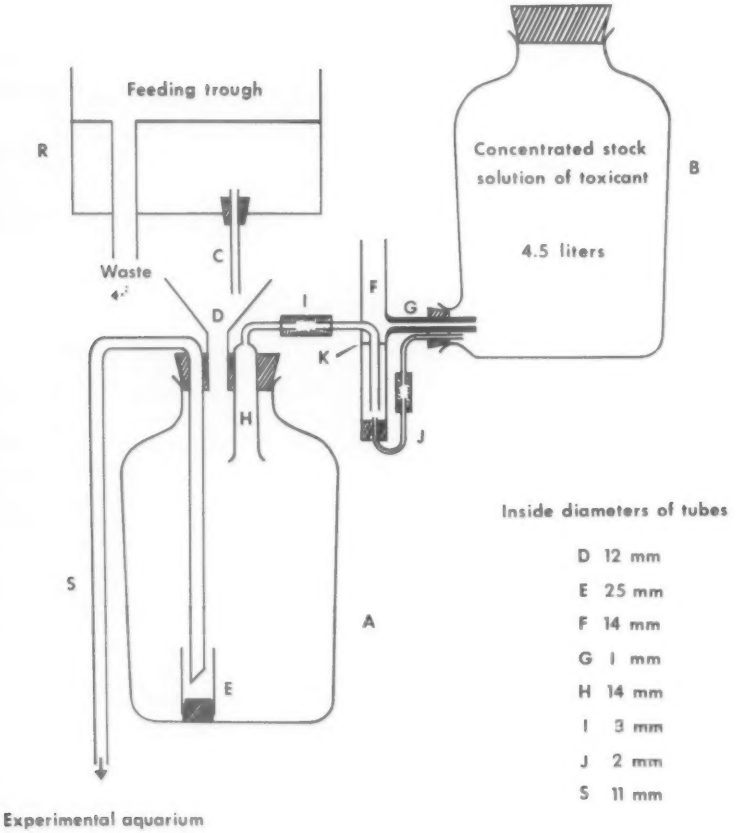

FIGURE 1.—Diagram of constant flow apparatus for toxicity experiments on fish.

aspirator bottle operating as a Mariotte bottle. The principle of the Mariotte bottle is the key to the operation of the system. The toxicant flows into reservoir *F* through the tube *J* but cannot rise beyond the level *K* because the bottle *B* being tightly stoppered, air enters only through the tube *G*. In the Mariotte bottle, the atmospheric pressure at the level *G* is equal to the combined pressure of air and liquid inside the bottle. Thus, the toxicant will rise no higher than this level in the reservoir *F*. In fact, owing to the action of surface tension in the tubes emerging from the bottle, the liquid does not quite reach the level *G* but remains at *K*.

Operation

In operation, siphon bottle *A* fills up with the dilution water. When the water reaches the lower level of tube *H*, the entrapped air is bubbled out through the solution in reservoir *F*. As the siphon is primed and bottle *A* empties, the content of reservoir *F* is drawn into bottle *A*. The diameter of tube *H* must be large enough so that the column of water will break when the level reaches the lower end of the tube. While bottle *A* empties and fills

again, the level of the stock solution in reservoir *F* is restored to its original position *K* and another cycle is repeated.

To calibrate the apparatus, the rate of flow of the feeding tube is measured with a graduated cylinder and a stopwatch. Or, alternatively, the time and the volume delivered in a complete cycle can be measured. Mariotte bottle *B* is filled with water and the apparatus is allowed to run for about 24 hr. The spent water in bottle *B* is then measured by replacement. A stock solution that will give, on dilution, the desired concentration in the experimental aquarium can then be prepared accordingly.

While the experiment is in progress, the stock solution is renewed as needed. This is done after closing *F* with a rubber stopper and *I* with a pinch clamp. Dimensions of the apparatus can be so arranged that a filled bottle *B* will last for about 48 hr. In the case of an insoluble toxicant which is in suspension, the danger of change in concentration due to deposition can be mitigated by using smaller amounts of the suspension and replenishing more frequently. From time to time, proper operation can be checked rapidly by measuring the time of a cycle and the volume of the stock solution used in 24 hr.

Before starting a new experiment or after about a month of operation the apparatus must be cleaned and may need recalibration. Every week or so, the feeding tube *C* is cleaned in place with a thin brush such as a pipe cleaner.

Since the flow of the toxic solution is dependent on the time of a cycle, a variation in the flow of the dilution water will cause only a slight alteration of the final concentration. Even an accidental stoppage of water will not necessarily spoil the experiment as the flow of the stock solution will stop as well.

Two batteries of 6 of these apparatuses have been in use satisfactorily in this laboratory for over a year. Aquaria used are plexiglass or ordinary plate glass 100-l aquaria. Two troughs *R*, each delivering dilution water to 6 aquaria are fed by gravity from a 400-gal stainless steel refrigerated tank. A float valve controls the level of tap water in the tank. Cooling is provided by a 5-hp refrigerating unit. Warming is accomplished by admitting hot water into the tank. An open-and-shut motor valve connected to the hot water line thermostatically regulates the inlet of hot water. In either warming or cooling, temperature can be controlled to within 0.2°C. The main disadvantage of this arrangement is that experiments at different temperatures cannot be run simultaneously.

On the other hand, it is particularly well adapted for such purposes as determining threshold toxicity values, comparing relative toxicity of poisons, and simulating in artificial streams the actual dilution of pollutants in rivers.

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THE OPERATOR'S CORNER

3. SEWAGE PUMPING AND PUMPING STATIONS*

Pumps serve many purposes in sewage collection systems and treatment plants. They may be classified by the character of the material handled: raw sewage, grit, trickling filter effluent, activated sludge, raw sludge, digested sludge, etc. Or they may relate to the conditions of pumping: high lift, low lift, recirculation, high capacity, etc. They may be further classified by principle of operation such as centrifugal, pneumatic ejection, diaphragm, or plunger.

The type of material to be handled and the function or required performance of the pump vary so widely that the designing engineer must use great care in preparing specifications for the pump and its controls. Similarly, the operator must conduct a maintenance and management program adapted to the peculiar characteristics of the equipment.

Designers and operators now place great importance on dependability of service, safety aspects, and automatic controls for all types of pumping stations. Most stations today have standby pumps to maintain continuous service when even the largest pump is out of service temporarily. Explosion-proof equipment, separation of wet wells containing sewage from areas containing possible sources of ignition, and ventilation facilities, manually and automatically controlled, are commonly installed. In most large stations and even in some small ones, flow-recording equipment is furnished to make an accurate record of instantaneous pumping rates and the total flow pumped.

Completely prefabricated pumping stations are gaining popularity, particularly as auxiliary lift stations.

3.1 DESCRIPTION OF EQUIPMENT

3.11 Centrifugal Pumps

A centrifugal pump (Figure 3-1) is fundamentally a very simple device, consisting of only one moving part, an impeller rotating in a casing. The impeller is supported on a shaft which is in turn supported by bearings.

When the pump speed is constant, the discharge will vary as the diameter of the impeller; the head which the pump will develop will vary as the square of the diameter; and the horsepower requirements or size of motor or engine will vary as the cube of the diameter.

The discharge of a given pump also may be increased by increasing the impeller speed. For example, if a 4-in. centrifugal pump discharges 150 gpm at 35-ft head with a 3-hp motor running at 1,350 rpm, it will discharge about 195 gpm at the same 35-ft head when the speed is 1,750 rpm.

The discharge, power, and head characteristics of any pump can be determined from the performance curve of the pump. This information is always available from the manufacturer.

* Chapter 3 of MOP 11, *Operation of Sewage Treatment Plants*. Chapters 1 and 2 appeared in the September issue.

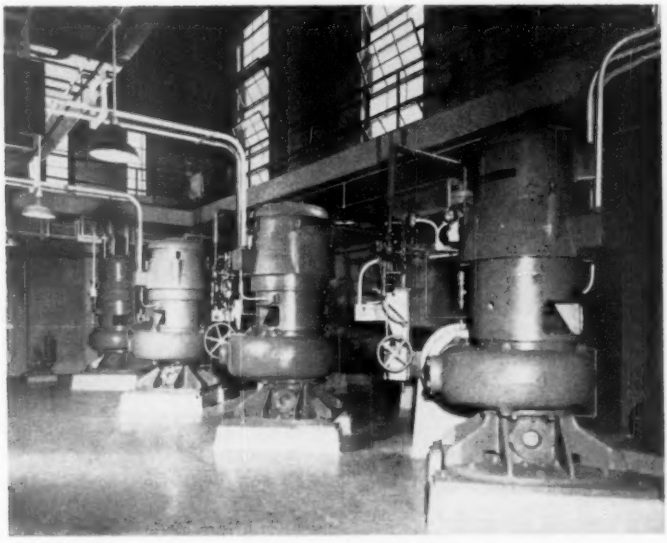

FIGURE 3-1.—Electrically driven centrifugal pumps.

The horsepower requirements of a centrifugal pump may be calculated as follows:

$$\text{Theoretical hp} = \frac{D \times H}{3{,}960}$$

where

D = discharge in gpm, and
H = total head in feet including friction.

The actual hp requirement equals the theoretical hp multiplied by the efficiency of the pump and driving unit.

The total head (or total dynamic head, TDH) is found by adding the friction loss in the pipe lines, valves, etc. (expressed in feet of water) to the vertical distance between the water level in the wet well on the suction side of the pump and the highest point of discharge. For example: the water elevation in the wet well is 20 ft below the point of free discharge at the settling tank and the friction loss is found to be 3 ft. The total dynamic head is then 23 ft.

Centrifugal pumps designed for pumping sewage usually have smooth channels and impellers with large-sized openings to prevent clogging.

Impellers may be of the open or closed type, but most pumps are provided with the two-blade enclosed type. Single-blade types, however, are used successfully. When open-type impellers are used, the blades are chamfered to minimize the surface area adjacent to the casing.

3.12 Pneumatic Ejectors

Ejectors have been used for many years in small installations to pump unscreened sewage and sludge. They are operated by compressed air, normally furnished by an electrically driven compressor located near the ejector. The ejector usually consists of a cast iron chamber with inlet and discharge pipes and either a float- or electrode-operated control system. After the chamber fills, compressed air enters and forces the sewage through the discharge piping.

3.2 NORMAL OPERATION

3.21 Control of Pump Rates

For the most effective operation of the wet well, pumping equipment, and treatment units, the pumps should be sized and operated so that their rate of discharge is nearly equal to the rate of inflow to the wet well. Careful selection of pumps, their cutin and cutout water elevations, and their operating sequence are among the adjustments which can be made to minimize wear on equipment and controls and improve treatment efficiencies.

By analysis of rate-of-flow charts and by visual observations of the pumping operations, the operator can determine which adjustments should be made. No single adjustment in the plant has as much effect on total plant performance as the rate of sewage flow through the settling tanks and the units which follow it. By careful selection of the pumps and adjustment of their controls, the operator usually can keep the pumping rate within 20 per cent of the rate of inflow to the station.

3.22 Protection of Pumps

For dependable and efficient performance, pumps require protection from abrasive materials and objects which may be damaging or obstruct the flow in the moving parts and suction and discharge lines. Most sewage, even from well-constructed separate sanitary sewer systems, contains substantial quantities of sand and other gritty material. Thus, where practicable, sewage pumps should follow a grit removal device. Pumps also should be preceded by facilities and devices for removal of such materials as rags, leaves, grass, wood, tramp metal, and other debris which may clog or damage pumps, lines, or valves.

3.221 Size and Shape of Wet Well:—If the raw sewage wet well is too large, solids will be detained unduly and decomposition will begin. This condition may create objectionable odors and interfere with later treatment processes. The detention time in the wet well may be reduced by adjusting the pump controls so that the drawdown water depth between the cutin and cutout points is lessened. If this does not reduce the detention time sufficiently, the cross-sectional area of the wet well can be reduced by constructing a partition wall of suitable material.

Sometimes the slope on the floor of the wet well is so flat that solids, including grit, cannot be removed by pumping. Frequent hosing will help keep this condition under control, but often the only effective remedy is reconstruction of the floor to provide sufficiently steep slopes toward the pump intakes.

3.222 Grease Control:—Grease should not be allowed to collect on the walls of the wet well. If it does, accumulations eventually will break loose in large pieces and disrupt operations by clogging the pump-suction line or the pump itself, or prevent the pump-control float from operating properly within the float-guide chamber or cage.

To control grease accumulations, walls of the wet well should be washed with a hose as often as necessary. Complete dewatering of the wet well is one way to remove most of the grease from the water surface.

3.223 Grit Control:—When a new sanitary sewer system is built and no grit chamber is provided, large quantities of sand may be deposited in the wet well. The sand should not be pumped out with a sewage pump; rather it should be removed with buckets or other suitable method as often as inspection indicates the need. If the sand is pumped, maintenance problems may be expected. Under these conditions, there is a good chance that the sand will get into the stuffing-

box packing if effective water seals are not used and promptly damage the packing and pump-shaft sleeve. On submerged pumps, the lower sleeve bearing may wear out in a few weeks under these conditions.

3.224 Ventilation:—Each year explosions take place in sewage wet wells. But in spite of severe property damage and loss of life many operators are still unimpressed about the need for extreme caution. Neither good ventilation, explosion-proof fixtures, nor complete separation from areas housing motors, etc., can protect against explosion, asphyxiation, or toxic effects if the operator is not careful.

The principles outlined in a later chapter on safety should be thoughtfully studied and considered.

3.23 Pneumatic Ejectors

The simplicity of an ejector installation makes operation and maintenance relatively easy and simple compared with a station containing a sewage wet well and centrifugal pumps. The principal units requiring regular inspection and occasional maintenance are the air compressor, flow-control devices such as floats and electrodes, and the check valves on the inlet and outlet lines.

Safety requirements are reduced to ordinary precautions common to areas containing electrical equipment by reason of the complete enclosure of the sewage during the entire pumping operation.

3.24 Centrifugal Pumps

3.241 Water Seals:—Water seals for packing glands should be provided on all sewage pumps. They serve to cool and lubricate the packing, and, for suction lifts, the seal prevents the entrance of air. Seals should be operated at approximately 2 to 5 psi (4.6 to 11.5 ft of head) greater than the pump discharge pressure. If the available water pressure is too high, a reducing valve may be used; if the pressure is insufficient, a water-seal pump may be required to boost it. To avoid a cross-connection, seal water should never be taken directly from a drinking water supply.

Packing-gland boxes should be watched for leakage of sealing water. A leak from the packing box while the pump is running is necessary to keep the packing cool and in good condition. But if there is too much leakage or no leakage, the gland should be tightened or loosened as required. Do not draw glands too tight, however. After adjusting packing glands, the shaft should be turned by hand to be sure it rotates freely. If excessive leakage continues, the packing probably is either burnt, worn, or dried out or the shaft has been scored. All packing should be renewed on a regular schedule to prevent scoring of the shaft or shaft sleeve.

When mechanical seals are replaced, the manufacturer's recommendations should be followed carefully.

3.242 V-Belt Drives:—When V-belt drives are used, sheaves must be in perfect alignment, for a slight variation will materially shorten the life and service of the belt, and at the same time increase the power consumption. Adjustments should be such that the belt is just tight enough to carry the load without slipping. In the case of some variable-speed drives, however, the proper belt tension is automatically maintained.

3.243 Pump Lubrication:—Pumps, motors, and drives should be oiled and greased in strict accordance with the recommendations of the manufacturer. Cheap lubricants may often be the most expensive in the end. Oil should not be put in the housing while the pump shaft is rotating because the rotary action of the ball bearings will pick up and retain a considerable amount of oil which will drain down when the unit comes to rest, resulting in an overflow of oil around the shaft or out of the oil cup.

3.244 Starting a Pump:—Before starting, a pump should be lubricated according to the lubrication instructions. The shaft should be turned by hand to see that it rotates freely, after which a check should be made to see that the shafts of the pump and motor are aligned and the flexible coupling adjusted. If the unit is belt driven, sheave alignment and belt adjustment should be checked. The current characteristics should be checked with the motor characteristics and the wiring should be inspected. Thermal units in the starter should be checked to be sure that they are set. Motor rotation should be determined by momentary contact to be certain that the motor will turn the pump in the direction indicated by the rotational arrows on the pump. If separate water seal units or vacuum primer systems are used, these should be started. Finally, it must be made certain that all valves in the suction and discharge lines are open. Sometimes there is an exception (below) in the case of the discharge valve.

A pump should not be run without first having been primed. To prime a pump, the pump must be completely filled with water or sewage. In some cases, automatic primers are provided. If they are not, it is necessary to vent the casing and most pumps are provided with a valve to accomplish this. The trapped air should be allowed to escape until water or sewage flows from the vent, after which the vent cap should be replaced. In the case of suction-lift applications, the pump must be filled with water unless a self-primer is provided. In nearly every case, it is permissible to start a pump with the discharge valve open. Exceptions to this, however, are where water hammer or velocity disturbances might result, or where the motor does not have a sufficient margin of safety or power. Sometimes there are no check valves used in the discharge line. In this case, it is necessary, of course, to start the pump and open the discharge lines afterwards. Where there are common discharge headers, it is essential to start the pump and open the discharge valve later.

After starting the pump, again check to see that the direction of rotation is correct. Packing-gland boxes should be observed for slight leakage as described earlier. Check to see that the bearings do not overheat from over- or under-lubrication. The flexible-coupling should not be noisy; if it is, it may be caused by misalignment or improper clearance or adjustment. Inspect piping for leaks.

3.245 Pump Shutdown:—When shutting down a pump for a protracted period, the motor-disconnect switch should be opened, and all valves on suction, discharge, and water-seal lines should be shut tightly and the pump completely drained by removing the vent and drain plugs. Sludge should not be permitted to remain in pipes or piping for any length of time; cases are on record where the gas produced has ruptured pipes and sludge pumps.

It is also a good policy to inspect the pump and bearings thoroughly so that all necessary sevicing may be done during the inactive period. The bearing housing should be drained and subsequently replenished with fresh oil.

3.246 Packing:—To renew the packing, remove the packing nuts, clamps, and glands in accordance with the manufacturer's instructions. This is often facilitated by the use of split glands. Pick out the old packing and the water-seal lantern ring from the packing box. Thoroughly clean the box and shaft. After this is done, cut the rings of packing so that the ends just meet around the shaft. Use the type of packing previously used. Each ring should be dipped in heavy oil before inserting it in the packing box. The new packing should, of course, be kept clean. One ring of packing should be installed at a time, slipping and tapping it into place. Be sure to place

the correct number of packing rings in front of and in back of the lantern ring so that it will line up with the water-seal inlet. This may be determined by noting the arrangement at time of removal or by depth measurement. Be sure ring joints are staggered to ensure a perfect seal. The reinstalled glands are then taken up lightly on the adjustment nut to form packing in the packing box. The gland followers should enter the packing box at least ⅛ in. to prevent blocking of the glands; but they must not jam or touch the shaft. The packing glands then should be adjusted as described previously so that they do not score the shaft. In the case of a water-seal unit, they should be adjusted so that there is a slight leakage.

3.247 Float and Electrode Switches:—To ensure the best operation of the pump, a systematic inspection of the water level controls should be made at least once a week. Check to see that:

1. The controls respond to a rising water level in the wet well,
2. The unit starts when the float switch or electrode system makes contact and that the pump empties the wet well,
3. The motor speed comes up quickly and is maintained, and
4. A brush-type motor does not spark profusely in starting or running.

3.248 Pump-Driving Equipment:—Driving equipment used to operate pumps include electric motors and internal combustion engines. In rare instances, pumps are driven with steam turbines and steam engines.

In all except the large installations, electric motors are used almost exclusively, with synchronous and induction types being the most common. Synchronous motors operate at constant speeds and are used chiefly in large sizes. The three-phase, squirrel-cage induction motors most often are used in treatment plants. These motors require little attention, and under average operating conditions the factory lubrication of the bearing will last approximately one year. In lubricating motors, it should be remembered that too much grease may cause bearing trouble or damage the winding.

3.248 Electrical Controls:—A variety of electrical equipment is used to control the operation of sewage pumps and protect electric motors. The simplest type of control unit consists of a counterweighted float which actuates a switch. When the float is raised by the sewage to a predetermined level, a switch is tripped which starts the pump; and when the sewage level falls to the cutoff level, the float switch stops the pump. The time required for each cycle and the length of time between cycles depends on the pumping rate and the quantity of sewage flow.

Where starters, disconnect switches, and cutouts are used, they should be installed in accordance with the local regulations regarding this equipment. In the case of larger motors, the power company often requires starters which do not overload the power lines.

The electrode-type, bubbler-type, and bladder-type water-level control systems are all similar in effect to the float-switch system.

3.249 Repair Shop:—Many large plants have fully equipped machine shops staffed with competent mechanics. But for smaller plants, adequate machine shop facilities often can be found in the community. In addition, most pump manufacturers maintain pump repair departments where pumps can be fully reconditioned.

The pump-repair shop in a large plant commonly includes such things as welding equipment, lathes, a drill press and drills, a power hack-saw, flame-cutting equipment, micrometers, calipers, gages, portable electric tools, grinders, a forcing press, metal-spray equipment, sand-blasting equipment, an arbor press, pneumatic tools, chippers, and coupling pullers.

3.25 General Operating Hints

A pump never should be throttled by closing the valve on the suction line. It should not be run longer than necessary with its discharge valves closed, as friction may cause the water in the pump to boil and damage the packing and other internal parts. If the discharge valve is to be opened after the pump is in operation, it should be opened slowly. Closing the discharge valve before stopping the pump is occasionally advisable to prevent pipe strains or water hammer.

3.3 OPERATING TROUBLES

The following list of operating troubles includes most of the causes of failure or reduced operating efficiency. The remedy or cure is either obvious or may be inferred from the description of the cause.

3.31 Symptom A—Pump Will Not Start

3.311 Causes:—

1. Blown fuses attributed to:
   (*a*) Rating of fuses not correct,
   (*b*) Switch contacts corroded or shorted,
   (*c*) Terminal connections loose or broken somewhere in the circuit,
   (*d*) Automatic control mechanism not functioning properly,
   (*e*) Motor shorted or burned out,
   (*f*) Wiring hookup or service not correct,
   (*g*) Switches not set for operation,
   (*h*) Contacts of the controls dirty and arcing,
   (*i*) Fuses or thermal units too warm,
   (*j*) Wiring short-circuited, or
   (*k*) Shaft binding or sticking by reason of rubbing impeller, tight packing glands, or clogging of pump.
2. Loose connection, fuse, or thermal unit.

3.32 Symptom B—Reduced Rate of Discharge

3.321 Causes:—

1. Pump not primed,
2. Mixture of air in the sewage,
3. Speed of motor too low,
4. Improper wiring,
5. Defective motor,
6. Discharge head too high,
7. Suction lift higher than anticipated,
8. Impeller clogged,
9. Discharge line clogged,
10. Pump rotating in wrong direction,
11. Air leaks in suction line or packing box,
12. Inlet to suction line too high, permitting air to enter,
13. Valves partially or entirely closed,
14. Check valves stuck or clogged,
15. Incorrect impeller adjustment,
16. Impeller damaged,
17. Packing worn or defective,
18. Impeller turning on shaft because of broken key,
19. Flexible coupling broken, or
20. Loss of suction during pumping caused by leaky suction line, ineffective water or grease seal, too high a suction lift or air or gas in the sewage.

3.33 Symptom C—High Power Requirements

3.331 Causes:—

1. Speed of rotation too high,
2. Operating heads lower than rating for which pump was designed, resulting in excess pumping rates,
3. Check valves open, draining long force main back into well,

4. Specific gravity or viscosity of liquid pumped too high,
5. Clogged pump,
6. Sheaves on belt drive misaligned or maladjusted,
7. Pump shaft bent,
8. Rotating elements binding,
9. Packing boxes too tight,
10. Wearing rings worn or binding, or
11. Impeller rubbing.

3.34 Symptom D—Noisy Pump

3.341 Causes:—

1. Pump not completely primed,
2. Inlet clogged,
3. Inlet not submerged,
4. Pump not lubricated properly,
5. Worn impellers,
6. Strain on pumps caused by unsupported piping fastened to the pump,
7. Foundation insecure, or
8. Mechanical defects in pump.

3.4 RECORDS AND REPORTS

A flow-recording meter should be a part of most pumping installations. When a separate electric meter is used, electrical consumption should be recorded daily.

The records most valuable to the operator are (*a*) daily flow charts, (*b*) total daily flow (from totalizer); and (*c*) electricity used.

By examination of the flow charts and power consumption, the operator may determine:

1. Sewage flow rates at any particular time;
2. Output of each pump separately and in combination with other pumps;
3. Whether unusual operating conditions are occurring or have occurred (see discussion under Sections 3.32 and 3.33);
4. Whether adjustments should be made to regulate rates of flow by changing pump sequence, eliminating or replacing one of the pumps, changing cutin and cutout water elevations, or changing cross-sectional area of the wet well; and
5. Quantity of sewage pumped per kilowatt hour. This is particularly valuable when comparisons are made over a long period to determine the change in pump efficiency. Rising power consumption per gallon pumped may be caused by any of the troubles discussed under Section 3.33.

The flow chart should be kept for ready reference and retained for several years. Total daily flow and maximum and minimum flow rates should be recorded for the main pumping stations along with other plant data.

4. SCREENING

Raw sewage contains materials such as rags, metal, garbage, sticks, and bricks. Unless removed ahead of all plant units, they may obstruct pipelines, channels, ports, pumps, orifices, and nozzles. They interfere with moving parts such as chains, sprocket wheels, and shafts, or occupy space needlessly in the treatment structures. Once admitted to the plant, they are difficult to remove and may require temporary dewatering or shutdown of critically needed treatment units. The purpose of screening devices is to remove these objectionable or damaging materials.

4.1 DESCRIPTION OF PROCESS

Screening facilities may be classified broadly as (*a*) coarse screens or racks, (*b*) bar screens, and (*c*) comminuting devices.

4.11 Coarse Screens or Racks

Coarse screens principally are used to remove objects such as large pieces of wood, bricks, or other floatables which may be washed into a combined sewer system (Figure 4-1). In such a system, these screens protect pumps and grit chambers which may be installed ahead of the regular bar screens or mechanical shredders.

The screens are constructed with evenly spaced rectangular or round bars set in a channel. The clear opening is usually from 2½ to 4 in. and the bars are sloped at an angle of from 45 to 60 deg from the vertical, terminating at a horizontal platform. A rake is used to remove objects from the rack.

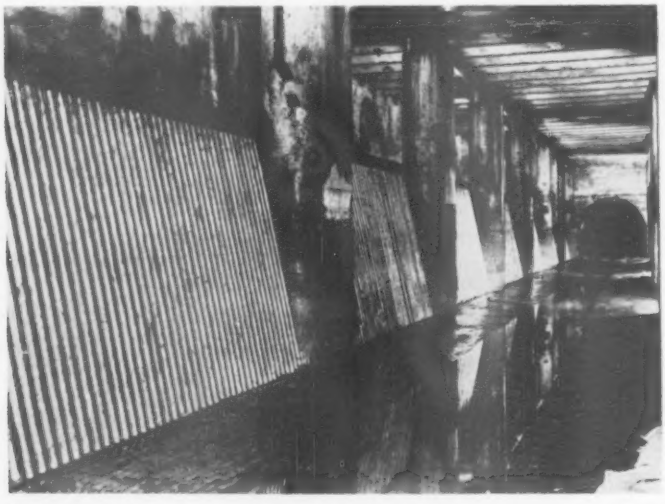

FIGURE 4-1.—Trash racks ahead of storm-water discharge on a large, combined sewer system.

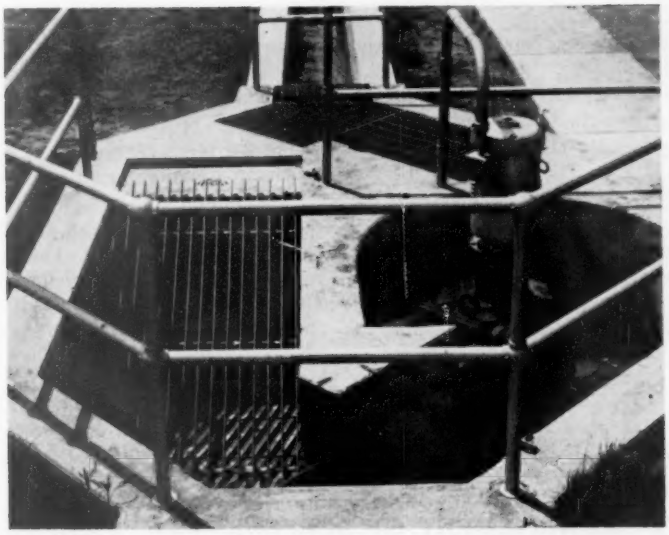

FIGURE 4-2.—Comminutor and standby bar screen.

4.12 Bar Screens

A bar screen is installed in nearly all plants as either the only screening unit or as a standby for a comminuting device (Figure 4-2). Hand-cleaned screens consist of a series of bars evenly spaced to form a rack. The slope of the rack is often 60 deg from the horizontal, with the unit terminating at a platform onto which the screenings may

FIGURE 4-3.—Large, mechanically cleaned racks.

be raked for dewatering. Openings between the bars are commonly between 1 and 2½ in.

When bar screens are used at large plants as the primary screening device, they are usually cleaned mechanically (Figure 4-3). The bars may be set vertically or at a small angle from the vertical. Screenings are removed by traveling rakes and deposited either on a platform or in buckets, cans, or trucks. Mechanisms may be operated continuously or intermittently.

4.12 Comminuting Devices

Many plants are now equipped with mechanical devices for cutting or shredding the materials normally classified as screenable (Figure 4-2). One common arrangement is to have an alternate channel with a bar screen so that screenable objects may be removed when the mechanical device is out of service for maintenance or repair.

A shredder or comminutor is essentially a slotted drum revolving in the path of the sewage. As the drum revolves against cutting surfaces, the materials which have passed through the slotted openings are cut into small pieces. Some of these devices have a stationary drum with cutting blades revolving against the slotted surfaces; others have moving blades which cut the materials collected on the screen.

4.2 MANAGEMENT OF SCREENING OPERATIONS

4.21 Coarse Screens

Daily removal of the debris collected on these racks normally is sufficient during dry weather. But when there is much storm water, a large number of objects may be washed down the sewers, requiring more frequent removal. Often this material may be hosed off thoroughly and hauled to a disposal site for municipal rubbish.

4.22 Bar Screens

Screenings should be removed as often as is necessary to prevent interference with reasonably free flow in the sewer; this may be from two to five times daily. If screenings are removed mechanically, the mechanism should be operated frequently enough to keep the channel or sewer flowing freely.

Mechanisms, chains, sprockets, and all other moving parts require lubrication and adjustment as recommended by the manufacturer.

4.23 Comminuting Devices

These devices usually are operated continuously. Sewage on the upstream side of the comminutor should be bypassed through the bar screen for a short period every few days to facilitate removal of settled material which, if permitted to flow through the cutting machine, would dull the cutting surfaces. Stone, sticks, and other unwanted material should be removed whenever observed.

As with all mechanical equipment, comminutors should be lubricated and otherwise maintained in accordance with the detailed instructions of the manufacturer. The cutting surfaces require sharpening, and the clearances require adjustment from time to time.

4.24 Disposal of Screenings

Objects collected on a bar screen are removed with rakes. Tines of hand rakes should be spaced to remove materials clinging to or partially wrapped around the bars. Sometimes operators dispose of the screenings by merely raking back and forth to dislodge material and force it through the spaces between the bars and into the flowing sewage. This defeats the purpose of screening. Good practice calls for care-

fully raking the stranded material upward to the platform for dewatering through the drain usually installed there. Hosing the screenings helps to remove some of the organic material and makes their handling and disposal less offensive. Disposal by burial is most common, and except for incineration or shredding it is the only satisfactory method.

Screenings removed mechanically from bar screens are usually deposited directly into cans or trucks for disposal. But sometimes they are deposited by the mechanism on a drained floor where they are dewatered before removal to the ultimate disposal site.

Frequent hosing of all floors and walls is necessary to keep the area clean and reasonably free of odors and flies. Cans, if they are used, should be covered.

A record is usually kept of quantities of screenings disposed of in large installations where handling or incineration operations are substantial.

5. GRIT REMOVAL

Grit comprises sand, dust, stones, cinders, and other heavy, inorganic material. If efficient removal is practiced, grit should contain little organic material and can frequently be disposed of as fill in the vicinity of the plant. The early removal of this material (*a*) prevents abrasive action on mechanical equipment such as pump impellers, packing glands, and collector mechanisms, (*b*) reduces hazards of clogging in plant units such as pipelines, tanks, and hoppers, and (*c*) simplifies sludge handling and disposal.

Grit removal usually follows or is part of the screen unit.

Grit is held in suspension in the raw sewage because the velocity in the sewer is high enough to keep it from settling. Separate sanitary sewer systems are usually designed to provide a velocity of at least 2 fps when flowing full. Combined sewer systems usually permit settling of a portion of the grit and some of the organic material during dry-weather flow, the deposited material being flushed out during periods of high runoff, when the velocity is 2 fps or higher.

When the velocity is reduced to about 1 fps, as in a grit chamber, differential sedimentation will occur; most of the grit will settle while most organic material will remain in suspension.

Grit varies over a rather wide range in size and general physical characteristics, depending on the source of material. It may originate in street surfaces and enter the sewer through street drains, storm sewers, or foundation drains. Where joints permit it, and the ground-water level is above the pipe, coarse sand may be admitted in substantial amounts.

5.1 DESCRIPTION OF PROCESS

5.11 Types of Facilities

Grit removal facilities are built in a variety of forms and shapes. Some are hand cleaned; others are equipped with mechanically operated grit removal equipment. The structures may consist of two or more parallel, narrow, relatively shallow channels, or square or circular tanks. Some depend on gravity only and others use air or impellers to aid in separation and removal. Some provide for washing the grit during removal; others simply remove it. All types require some means for controlling velocity and settling time.

5.12 Hand-Cleaned Chambers

Hand-cleaned grit chambers are commonly used in small plants. They usually consist of two parallel elongated channels equipped with velocity control devices to provide a nearly constant velocity of about 1 fps for the expected range of flow rates (Figure 5-1). Devices commonly used for velocity control are Parshall flumes, parabolic flumes, and proportional weirs.

Many installations have hopper-shaped bottoms below the flow line for grit storage. Frequently, drains are installed in the bottom of the hoppers to dewater the channel to facilitate removal of the grit. Grit is removed by hand to buckets or wheelbarrows using shoveling hoes or shovels.

5.13 Mechanically Operated Chambers

Mechanically equipped grit removal facilities usually are installed where

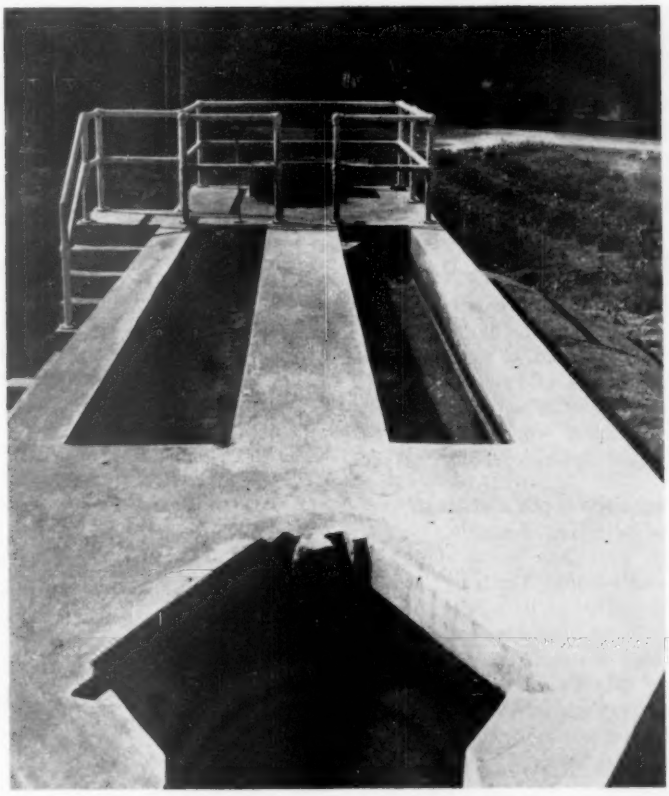

FIGURE 5–1.—Parallel grit chambers.

(*a*) the flow is relatively high, (*b*) a large quantity of grit is expected, or (*c*) the chamber is installed so far below ground surface that hand removal is impracticable.

Two or more chambers are usually installed to provide needed flexibility in velocity control and detention periods for widely varying rates of sewage flow, particularly for combined sewer systems. Multiple chambers also permit continuous operation when one of the units is temporarily out of service for equipment repair or maintenance.

5.131 Mechanical Equipment:—Grit is removed continuously by equipment which scrapes or pushes it onto some form of conveyor or which delivers the grit to a receiver such as a can, storage bin, or truck. The grit is washed as it is moved through the sewage, thereby removing some of the organic material from the grit particles.

Other methods for mechanical removal utilize compressed air or an impeller to suspend the organic material while the grit settles for removal by pumping or conveyors.

5.2 NORMAL OPERATION

Although grit removal is a simple uncomplicated process, its effective control may require all the ingenuity and resourcefulness of the well-trained operator familiar with the peculiarities of the sewer system and characteristics of the sewage.

5.21 Manually Cleaned Units

With ideal operating conditions and a sufficient number of channels, effective grit removal can be accomplished while one or more additional channels are idle for dewatering and grit removal. Operation would then be reduced to the following procedures:

1. Measure the depth and location of grit in the collecting channels.
2. When grit has accumulated to about one-half liquid depth in the first half or two-thirds of its length, bypass the flow to the unused, cleaned channel.
3. Dewater channels containing grit by opening valves on dewatering lines. These lines may discharge to sand drying beds, raw sewage wet wells, or chlorine contact tanks.
4. Remove grit after it has been well dewatered, using shovels or shoveling hoes. Measure or estimate quantity removed. Grit may be buried in shallow trenches or hauled to sanitary land fill areas. Disposal on land surfaces may create objectionable odors.
5. Flush out channels and close valves on drains. The unit is now ready for service.
6. Analyze samples of grit removed for volatile solids. If an unusually high percentage is found, undertake corrective measures outlined in Section 5.31.
7. Check for unusual quantities of grit in the treatment units which follow grit removal.
8. Remove grit from other plant units if found in substantial quantities and undertake corrective measures outlined in Section 5.32.

5.22 Mechanically Operated Units

Effective operation of these units requires control procedures similar to those outlined for manually cleaned units. In addition, the following procedures and practices are required:

1. Maintain all mechanical equipment in accordance with manufacturer's instructions.
2. Keep all moving equipment free of debris and daily hose down all exposed walls, emergent scrapers, etc.
3. Dewater each unit at least once yearly during season when the smallest quantity of grit is expected. Examine all equipment and tank surfaces below the water line, and repair, adjust, or replace parts as needed.

5.23 Safety Measures

Explosive or toxic gases contained in the sewage have opportunity in grit chambers to mix with air causing toxic conditions or explosive atmosphere. Except when these chambers are completely exposed to the atmosphere, the following precautions are essential:

1. Grit chamber should be thoroughly ventilated at all times.
2. The area should be considered an explosive zone and protected as such.
3. The area should be considered a toxic zone and adequate precautions taken to insure that the operators are protected and periodically observed while working in the units.

5.3 OPERATING TROUBLE

5.31 Symptom A—Excessive Organic Material in Grit

5.311 Cause:—Velocity is too low and detention period too long.

5.312 Prevention and Cure:—Reduce cross-sectional area of channel occupied by the flowing sewage by:

1. Installing planks, bricks, or tile along sides of the channel;
2. Reshape or repair outlet weir to proportionally reduce depth of flow for all normal present flow rates;
3. Decrease number of channels used; or
4. Reduce length of channel by moving outlet weir.

5.32 Symptom B—Carryover of Grit

5.321 Cause:—Velocity is too high and detention period too short.

5.322 Prevention and Cure:—

1. Remove grit more frequently;
2. Increase number of channels used; or
3. Increase cross-sectional area of channels.

5.4 LABORATORY CONTROL

The following simple records and tests are useful to the operator, particularly for detection of unusual trends in quantity and quality of the grit. Comparison of the records from time to time may warn the operator of a new or changing source of grit and may help him and others to locate the trouble.

5.41 Quantity of Grit Removed

The volume of the grit removed should be recorded each time hand-cleaned units are emptied. Daily records should be kept for mechanically operated units. Quantity should be calculated in terms of the flow that has passed through the unit. For example, if 20 cu ft is removed and the total sewage flow through the unit has been 5 mil gal since the chamber was previously cleaned, the grit removed should be recorded as 4 cu ft/mil gal.

5.42 Volatile Solids Content

The per cent volatile matter should be determined regularly where laboratory facilities are available. Tests should be made daily for mechanically cleaned units and for each batch removed in hand-cleaned units.

6. SEDIMENTATION

Solid particles once suspended in sewage are carried forward because of the speed and mixing action of the liquid. If the velocity of flow and mixing action are greatly reduced, most of the solid material will settle. In sewage treatment, the purpose of sedimentation is to remove all settleable solids.

Primary settling tanks remove the grosser suspended load from sewage prior to discharge to receiving waters or secondary treatment units. Secondary or final settling tanks remove the material converted into settleable solids by biological or chemical treatment. Preaeration, mechanical flocculation, and chemical treatment are sometimes used to increase sedimentation efficiency.

Sedimentation tanks may be designed for fill-and-draw or continuous operation. Except for some industrial wastes, fill-and-draw operations are seldom used. Continuous-flow tanks are usually provided for the treatment of municipal sewage and in most instances for the treatment of industrial wastes.

6.1 FUNDAMENTALS OF REMOVAL

Particles in suspension may be classified as granular or flocculent. Granular particles (sand, silt) settle at constant velocity, independent of one another and without change in size, shape, or weight. Flocculent particles (organic matter, flocs formed by coagulants or biological growths) tend to cluster during settling, with changes in size, shape, and relative density. The clusters ordinarily settle more rapidly than individual particles. Settleable solids comprise that portion of the granular and flocculent material which settles under quiescent conditions in a reasonable time. This period is commonly, though arbitrarily, taken as one hour. Non-settleable solids are so finely divided that they will not settle, even under these conditions.

6.12 Factors Influencing Sedimentation

6.121 Sewage Characteristics:—Sedimentation is affected by the strength and freshness of the sewage and the density, shape, and size of the particles. Strong sewage usually settles more readily than weak sewage. On the other hand, stale sewage settles less readily than fresh sewage because the particle sizes are reduced by biological degradation, and also the particles tend to be buoyed by gas. A dense particle settles more rapidly than a light one; a particle with a large surface area, in relation to its weight, settles slowly; and one with irregular shape has greater frictional drag and settles more slowly than a particle with a regular shape.

6.122 Detention Period:—This is the time required for the sewage to flow through the tank at a given rate of flow. The detention period should be sufficient to allow practically complete removal of settleable solids. Long detention periods do not materially improve removal and may actually be harmful by allowing the sewage to become stale.

6.123 Tank Dimensions:—Rate of removal of granular particles settling at

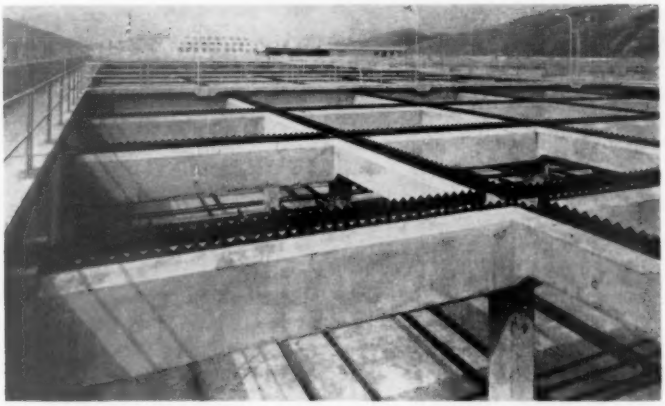

FIGURE 6-1.—Effluent weirs. Note sludge collector flights on floor.

uniform velocities depends almost entirely on tank surface area. Rate of removal of flocculent particles settling at variable velocities depends on both tank surface area and depth. In plain sedimentation, surface loadings of 900 to 1,200 gpd/sq ft and depths of from 8 to 10 ft are common.

6.124 Inlets and Outlets:—Inlets are designed to reduce entrance velocity and distribute flow uniformly throughout the cross section of the basin by means of suitable openings, baffles, or other means.

Outlets for removal of clarified sewage may be ports or weirs (Figure 6-1). They should be of sufficient area or length so that velocities at the outlet end of the tank are low enough to prevent carryover of bottom deposits. Baffles should be provided ahead of outlet weirs in primary tanks to prevent loss of floating solids and grease.

6.13 Grease Removal

In sewage, the fats, waxes, free fatty acids, calcium and magnesium soaps, mineral oil, and other non-fatty material are collectively called grease. Under quiescent conditions, a part of the grease settles with the sludge, and part floats to the surface where it may be removed by a suitable skimming device.

Flotation of grease can be hastened by blowing air into the sewage. The small, rising air bubbles attach themselves to the grease particles, increasing their buoyancy and entrapping them in the surface foam created.

6.14 Chemical Treatment

Chemical treatment is used when it is desired to remove more finely divided suspended and colloidal material than is possible with plain sedimentation. The chemicals used react with constituents of the sewage or with other chemicals added for this purpose to form a heavy, flocculent precipitate. As the precipitate settles, the suspended and colloidal particles are entrapped and adsorbed on the large and sticky surface. Removals lie between those obtained by plain sedimentation and those obtained by secondary biological treatment.

6.2 DESCRIPTION OF PROCESS

Sedimentation tanks may be classified into two types, (*a*) rectangular and (*b*) circular (or square). In the rectangular type, sewage flows from one end of the tank to the other and the sludge is moved by scrapers to the

inlet end. In the circular or square type, the sewage usually enters at the center and flows radially to the periphery, with the sludge pushed or otherwise transported to the center. There are some designs, however, in which sewage enters circular tanks tangentially at the outer edge and flows inward.

Small rectangular or circular tanks may have a sludge collection hopper covering the entire bottom of the tank in lieu of a sludge collection mechanism. Since hopper slopes must be steep, this type of tank is confined to small sizes to avoid excessive tank depth.

6.21 Rectangular Tanks

Rectangular tanks may be in single units or in a series of units. The length of the tank is usually several times the width. Deposited sludge is moved to a hopper at one end, either by wood flights mounted on parallel strands of conveyor chain or by single bottom scraper mounted on a carriage moving on rails fastened to the tank walls.

Chains and flights are carried on submerged sprockets, shafts, and bearings, and are driven by a motor. The flight may be arranged to remove floating material from the tank on their return path (Figure 6-2), or flat, water sprays inclined to the surface may be used instead. Moving carriages are towed by cables or powered by traction drives.

The settled solids may be collected in a single hopper, in multiple hoppers, or in a transverse trench with a hopper at one end. The transverse trench may be equipped with a flight-and-chain collector or a screw conveyor to carry the sludge to the hopper.

6.22 Circular Tanks

The term clarifier often is used to describe circular sedimentation tanks. The most common type of circular tank is a single structure in which the flow is introduced at the center into a feedwell which dissipates inlet velocities (Figure 6-3). The sewage moves radially from the feedwell to the weir and overflow trough at the outside edge of the tank. Settled solids are raked to a

FIGURE 6-2.—Rectangular sedimentation tanks.

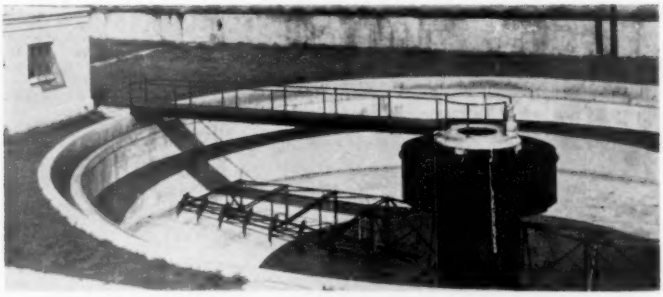

FIGURE 6-3.—Secondary settling tank with sludge scraper.

hopper near the tank center by arms attached either to a drive unit at the center of the tank or a traction unit operating on the tank wall. In primary tanks, floating material usually is moved to the skimmer by a surface blade attached to the sludge collector.

6.23 Flotation Processes

Grease may be separated by natural flotation under quiescent conditions. This may be done in sedimentation tanks or in separate skimming tanks. In rectangular sedimentation tanks, the floating material is moved to one end of the tank by sludge collector flights returning along the surface or by flat water sprays inclined to the surface. In circular tanks, floating material is usually moved to a skimmer (Figure 6-4) by a surface blade attached to the sludge collector. The collected material is removed from the tank through a slotted pipe or other type of skimming device.

Grease may be separated also by air flotation, a process which relies on the lifting action of small air bubbles. Air-flotation units now in use differ in the method of air application, source of air, and means for increasing utilization of the rising bubbles. Air may be introduced to the sewage before it enters the flotation tank, and sometimes a vacuum is applied to the tank surface to aid the process. Blowers, injectors, or mechanical aerators are commonly used.

6.231 Preaeration:—Preaeration by air diffusion or mechanical aeration can take place in separate structures (Figure 6-5) or in the settling tank itself as an aid to the sedimentation process. Several different designs are in use.

6.24 Chemical Treatment

6.241 Facilities:—Chemical treatment consists of a quick or flash mixing of the coagulants with the sewage, followed by slow mixing or flocculation (Figure 6-6), followed in turn by settling and removal of the sludge. Usually, the process follows screening and

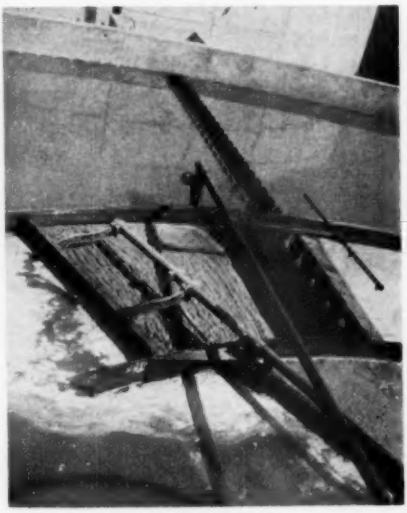

FIGURE 6-4.—Skimmer on a circular tank.

grit removal and precedes sedimentation; it may follow sedimentation in some cases, particularly where large quantities of readily settleable material are present or where oils and greases are removed earlier. The flash-mix time is very short, ranging from less than 1 min to 3 min. Flow-through time for slow mixing or flocculation is usually about 30 min, ranging from about 20 min to 1 hr.

6.242 Adaptation:—The process is generally employed where a degree of treatment intermediate between plain sedimentation and secondary biological treatment is needed. This process is often used under the following conditions:

1. Where wastes have a large proportion of finely divided suspended solids which will not settle readily by gravity alone;
2. For seasonal treatment of wastes such as cannery or other organic wastes, or to satisfy unusual stream requirements where installation of extensive secondary treatment is not justified; or
3. Where more than plain sedimentation is required, but the constituents are not amenable to biological treatment.

6.243 Chemicals Used:—Many chemicals are used for this process, the most common of which are:

1. Aluminum sulfate or alum,
2. Ferric sulfate and ferric chloride,
3. Ferrous sulfate and ferrous chloride, and
4. Lime, used alone or with alum or ferric salts.

6.244 Sludge Produced:—Much more sludge is produced in chemical treatment than in plain sedimentation by reason of the additional solids removed and the quantity of chemicals recovered. Some sludges are readily digested while others are not, depending on the chemicals used. Lime precipitation makes the pH and total alkalinity too high for favorable digestion.

FIGURE 6-5.—Preaeration unit.

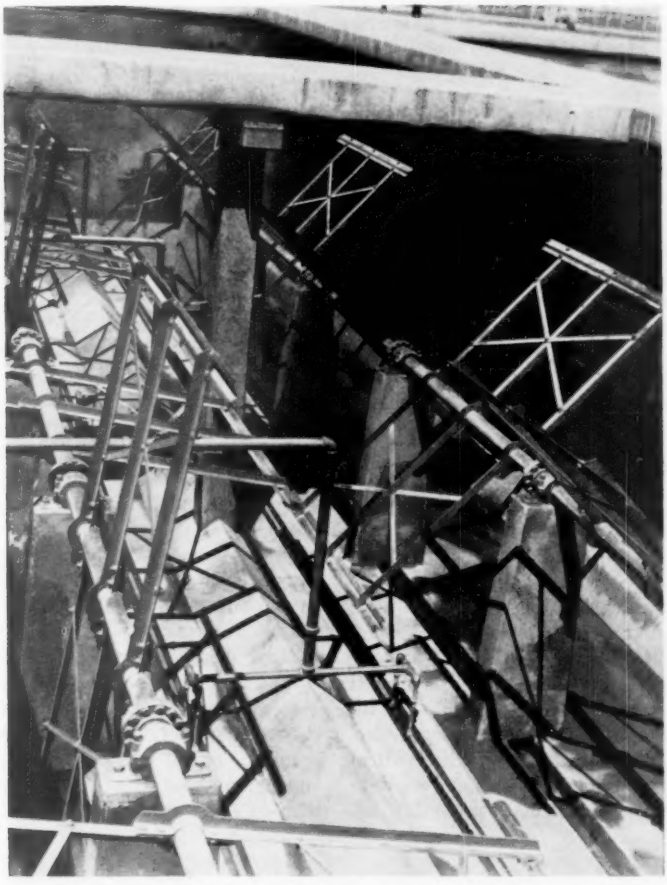

FIGURE 6–6.—Mechanical flocculator.

Sludges associated with ferric or ferrous coagulants will digest well. Chemical sludges are commonly dewatered on vacuum filters without digestion when chemical treatment is operated on a year-round basis.

6.3 NORMAL OPERATION

6.31 Equal Flow

Sedimentation is most efficient if the flow is distributed equally among all available settling tanks. Unequal flow causes uneven detention periods in the various tanks and always results in a lesser over-all reduction of suspended solids and BOD.

The overflow weirs must be kept clean and at equal elevations to maintain equal flow. To divide the flow equally, it is sometimes necessary to provide a control orifice at the entrance to the sedimentation tanks.

The purpose of the inlet baffle is to help distribute the sewage evenly. Baf-

fles should be clean and in sound condition; unless they are, reduced efficiency may result.

6.32 Routine Inspection

The settling tank and all mechanical equipment should be inspected several times during each shift. Sludge removal equipment, grease skimmers, and pumps should be checked for proper operation and conditions on the water surface should be observed. Floating sludge or an abundance of gas bubbles are evidence of equipment or process breakdown. Often, a breakage of underwater chains can be detected in this way.

6.33 Scum Removal

A scum barrier or baffle normally is provided somewhere in the flow path between the influent and effluent portions of the tank. The floating scum may be removed either mechanically or manually (Figures 6-4 and 6-7). There will be less carryover of scum, floating garbage, and grease if scum is removed before any appreciable amounts are allowed to accumulate on the scum barrier. Excessive skimming will result in too much water being carried over with the scum. Insufficient skimming allows scum to flow around or under the baffle and out with the effluent. In rectangular settling tanks without automatic skimmers, it is desirable to remove the tank skimmings manually after the flights have made one complete revolution. Ideally, small amounts of scum should be skimmed continuously. Each operating staff will determine by experience that pattern best suited to its plant.

6.34 Collecting Sludge

There are two methods in general use for moving settled sludge to a point in the settling tank where it may be drawn off. The simplest method is used only in small tanks where the whole bottom of the tank is built on a steep slope and the settled sludge slides down the slope to the point of drawoff. For this kind of a tank, the sloping sides should be scraped once each day to be sure the sludge moves to the bottom.

Most treatment plants use mechanical sludge collectors in tanks having a gently sloping bottom. The collectors may be run on varying schedules depending on the type of plant and equipment and the characteristics of the sewage. Many operators of rec-

FIGURE 6-7.—Manual skimming.

TABLE 6-1.—Characteristics of Raw Sludge from Plain Sedimentation Tanks

Quality	Description	Remarks
Physical texture	Non-uniform, lumpy	—
Color	Brown	Dyes from industrial wastes may color the sludge or it may be dark gray or black in color if septic (see septicity below)
Density	Average solids content of 3–8%	A good quality for drawing should average 5–7% solids
Odor	Usually offensive	May have little or no odor in presence of such industrial wastes as metallic salts
Volatile matter	Average range, 70–80% of total dry solids	Occasionally may run as high as 85% or as low as 60%
pH	Usual range 5.5–6.5, average value about 6.0	Affected by industrial wastes and septicity of sludge
Total alkalinity	Range 300–1,000 mg/l, usually 500–800	—
Mineral or non-volatile solids content	Around 30% of total dry solids	If appreciably more than 30%, presence of grit should be suspected
Septicity	Non-septic	Septic raw sludge is, however, rather common because of: (*a*) septic raw sewage, (*b*) retention of sludge too long in tanks, and (*c*) poor quality overflow liquor from digestion tanks
Sludge volume	From 70–700 cu ft/mil gal of sewage treated, average about 250–350 cu ft/mil gal	High volumes indicate too thin a sludge is being drawn or some voluminous industrial waste is present. Low volumes indicate weak sewage, poor tank efficiency, or sludge building up in tanks
Digestibility	Normally easily digested	Non-digesting raw solids indicate harmful industrial wastes, such as metallic salts or fibrous matter
Grease content	Normally 15–20 mg/l	Appreciably higher figures are indicative of industrial wastes such as waste oils or wool scouring wastes
Drainability on sand beds	Poor	—

tangular tanks start the collector mechanisms about an hour before sludge is drawn and turn them off when drawing is complete. A few operators let them run continuously, but normally this is not necessary. In a square or round tank, it may be necessary to run collectors continuously as it takes longer for sludge to be moved to the hoppers. This precaution avoids the possibility of overloading the mechanism by piling up of solids.

6.35 Raw Sludge Characteristics

The quality of sludge removed from plain sedimentation tanks varies widely from plant to plant. Variations depend upon many factors, but the principal ones are these:

1. Composition of the sewage entering the tank—freshness, strength, constituents, settleability;
2. Characteristics and adequacy of the settling tank; and
3. Management of the settling tank, including methods of sludge removal.

The observations of Van Kleeck (1) on the quality of raw sludge settled from sewage largely of domestic origin are summarized in Table 6-1.

6.36 Sludge Pumping

The proper solids concentration of sludge pumped from primary settling tanks will vary from plant to plant,

but will generally be from 5 to 8 per cent. The sludge pumped from secondary settling tanks is much thinner, and pumpage is usually dictated by the requirements of the biological treatment.

Pumping may be continuous or intermittent. When several pumps are used, one pump may withdraw sludge from one hopper while another pump withdraws from another; however, a single pump should not withdraw sludge from more than one hopper at a time. Differences in head losses in the piping and varying densities, viscosity, and clogging factors would cause too much sludge to be withdrawn from one hopper, and too little from the other. Pumping should be for short duration at frequent intervals rather than for long periods at infrequent intervals.

In small plants with operating personnel on duty only part of the time, sludge may be pumped only once or twice a day; in a large plant with operating personnel on duty at all times, sludge may be pumped hourly. Sludge can be pumped only as fast as it can flow to the sludge inlet. If pumping is too fast, settled sewage will be pumped to the digester instead of sludge.

6.37 Volume of Sludge

Excess liquid pumped to a digester needlessly occupies valuable digester capacity. With reasonable care, the operator should be able to remove sludge containing five per cent solids.

The volume of sludge to be removed from any settling tank may be estimated by the operator who measures the influent and effluent suspended solids, the total solids in the sludge pumped, and the sewage flow. In the following equations the method of computation is shown:

$$\text{dry solids removed (lb)} = \text{SS removed (mg/l)} \times 8.35 \times \text{sewage flow (mil gal)} \quad (6\text{-}1)$$

$$\text{wet sludge removed (lb)} = \text{dry solids removed (lb)} \times \frac{100}{\%\text{ dry solids in sludge}} \quad (6\text{-}2)$$

$$\text{wet sludge removed (gal)} = \frac{\text{wet sludge removed (lb)}}{8.35} \quad (6\text{-}3)$$

6.371 Example:—Assume sewage flow = 1.0 mil gal; SS in effluent = 250 mg/l; SS in effluent = 150 mg/l; dry solids in sludge, 5 per cent. How much sludge must be pumped?

By substituting in Eq. 6-1 it is found that the dry solids removed = 100 × 8.35 × 1.0 = 835 lb. Now substituting in Eq. 6-2, the wet sludge removed = 835 × 100/5 = 16,700 lb. Finally, by Eq. 6-3, the volume of sludge pumped = 16,700/8.35 = 2,000 gal.

If the solids in the sludge pumped had been 3 rather than 5 per cent, the resultant volume would have been 835 × 100/3 or 27,830 lb, or 3,333 gal.

From calculations of this kind, the operator may determine the length of time required for pumping with any pump of known capacity. Frequently, a little arithmetic of this kind will cut down the pumping of excess water to the digesters.

The wide variation in the sludge volume with a small change in the concentration of solids is shown in Table 6-2.

The careful operator who is able to pump sludge into a digester at 6 per cent solids must pump only half as many gallons as the careless operator who allows excess water to dilute the sludge to 3 per cent solids.

TABLE 6-2

Dry Weight of Sludge (lb)	Concentration of Solids (%)	Sludge Volume (gal)
900	3	3,600
900	4	2,700
900	5	2,160
900	6	1,800

6.372 Pumping Appurtenances:— Wherever a flushing system is provided, a few minutes' flushing once a week is usually adequate to keep the line clean. The lines also should be flushed if the sludge becomes difficult to pump. Cleanouts in sludge suction lines and sludge pumps often are provided and these should be checked if pumping trouble occurs. When the installation permits, sludge lines can be cleaned with hydraulically propelled cleaning tools or balls.

Vacuum and pressure gages should be installed on sludge pumps to indicate the conditions under which the pump is working. Pressure gages are useful also in indicating a stoppage or solids buildup in the pressure line. Gages can be added easily by the operator.

6.38 Miscellaneous Maintenance

Adoption of the following regular maintenance practices will not only increase efficiency, but provide better working conditions for the operator:

1. Remove accumulations regularly from the inlet baffles and effluent weirs. Generally, this can be done with a hose and broom. The necessary frequency can be determined only by experience.
2. Clean scum removal equipment regularly. This equipment is a major source of obnoxious odors and unsightly appearance when neglected.
3. Keep cover plates in place except when operations require their removal.
4. Hose down and remove all sewage and sludge spills as soon as possible.
5. Keep leakage from sludge pump glands off floor surfaces by piping it to floor drains.
6. Prepare a lubrication chart for all mechanical equipment. The requirements for each piece of equipment can be obtained from the manufacturer. Any reliable oil company usually can provide the services of a trained lubrication engineer to make recommendations.

Schedules for lubrication and other routine jobs should be posted on a check-chart system so that a record is kept of the date a job was done, who did it, and when it is to be done again.

The underwater portion of the sedimentation tank should be inspected regularly; once a year is usually sufficient. The best time for doing this depends on the weather, treatment requirements, and available personnel. Essentially, the following work is required for this operation:

1. Dewater the tank, discharging its contents to another tank whenever possible;
2. Inspect all mechanical equipment for wear and corrosion;
3. On rectangular tanks with mechanical sludge collectors, check wood flights, wearing shoes, main chains, drive chains, sprockets, guide rails, and grease skimmers;
4. Replace or repair all defective, broken, or badly worn parts;
5. Adjust chains and be sure flights have proper clearance at tank walls (at least 1 in.);
6. Check all metal work for corrosion, replace where advisable, and apply protective coating; and
7. Check concrete work and patch all defective areas.

If unusual corrosion of concrete or metal work is found, it may be necessary to provide ventilation for above-water facilities and institute a preventive program. Concrete surfaces may require a protective coating or neutralizing agent and metal may require cathodic protection or a protective coating. Engineering studies are often necessary to develop a satisfactory solution.

6.39 Interrelation with Other Processes

Management of a primary sedimentation tank is closely related in many ways with the operation of other plant

facilities. No other single unit provides as much opportunity for detection of conditions which affect the performance of other plant processes. By early detection of unusual characteristics, the operator frequently may adjust his normal methods of management of this facility so that harmful and deleterious effects on the later phases of treatment may be minimized.

6.4 OPERATION TROUBLES

6.41 Symptom A—Floating Sludge

6.411 Cause:—Sludge decomposing in tank and buoyed to surface.

6.412 Prevention and Cure:—Remove sludge more completely or more frequently by one or more of the following methods:

1. Operate mechanical scrapers for longer periods or more often;
2. Replace broken or warped wooden flights; or
3. Remove sludge from hoppers more completely by pumping or gravity withdrawal for longer periods or at slower rates. Where sludge clings to inclined surfaces, remove by scraping or jetting.

6.42 Symptom B—Contents Black and Odorous

6.421 Cause:—Septic sewage or strong digester supernatant.

6.422 Prevention and Cure:—
For septic sewage:

1. Disconnect all septic tanks from the collection system;
2. Modify, pretreat, reduce, or eliminate organic wastes, such as cannery, milk processing, brewery, tannery, and organic textiles wastes;
3. Chlorinate up sewer or ahead of sedimentation tank to delay or reduce decomposition of sewage; this is particularly effective when time of travel in sewer is unusually long or sewage temperatures are high, and when certain industrial wastes are present;
4. Preaerate wastes of types listed in No. 2 above; or
5. Improve hydraulics of sewer system to reduce accumulations of solids.

For strong digester supernatant:

1. Correct or improve sludge digestion to produce improved quality supernatant;
2. Reduce rate of withdrawal of supernatant to sedimentation tank;
3. Select better quality supernatant from another zone or delay withdrawal until quality improves if that is possible;
4. Discharge portion or all of supernatant until quality improves to other points such as lagoon, sludge drying bed, or aeration tank (under some circumstances, discharge to these points may be permanent); or
5. Presettle or elutriate the supernatant.

6.43 Symptom C—Excessive Sedimentation in Inlet Channels

6.431 Cause:—Velocity too low for cross-sectional area of channel at present flow rates.

6.432 Prevention and Cure:—

1. Reduce cross-sectional area by installing inner wall of suitable material along one wall of channel; or
2. Agitate with air, water, or other means to prevent deposition.

6.44 Symptom D—Excessive Fouling of Surfaces and Weirs with Sewage Solids or Growths

6.441 Cause:—Accumulation of sewage solids and resultant plant growth.

6.442 Prevention and Cure:—

1. More frequent and thorough scrubbing of all surfaces in contact with sewage; and
2. Prechlorination coupled with No. 1 above.

6.45 Symptom E—Intermittent Surging of Flow

6.451 Cause:—High intermittent pumping rates.

6.452 Prevention and Cure:—

1. Adjust controls for pump sequence, pump combinations, and cutin and cutoff water surface elevations to maintain pumping rates close to rates of inflow; or
2. Install or adjust baffling to reduce inlet velocities and to produce better flow distribution across entire width of tank.

6.46 Symptom F—Broken Scraper Chains and Frequent Shear Pin Failure

6.461 Cause:—Excessive load on mechanical sludge scraper.

6.462 Prevention and Cure:—

1. Periodically dewater tank and examine all metal parts for defects and wear;
2. Replace defective and worn parts, particularly cotter pins, worn chain links, and badly worn, broken, or warped wooden scrapers (flights);
3. Remove or effectively break up ice formation on walls and surfaces;
4. Install grit chamber or otherwise reduce entrance of grit to the sewer system if grit accumulation is substantial; or
5. Operate collector mechanism for longer period and pump sludge more often.

6.47 Symptom G—Sludge Hard to Remove from Hopper

6.471 Cause:—

1. High content of grit, clay, or other heavy or easily compacted material; or
2. Low velocity in withdrawal lines.

6.472 Prevention and Cure:—

1. Reduce grit content by installing grit chamber or locate and correct sources of grit entering the sewer system;
2. Reduce clay content by eliminating source where possible, particularly in separate sewer systems;
3. Loosen compacted material manually or by pressure jetting with air or liquid;
4. Back-flush clogged pipelines;
5. Pump sludge more frequently; or
6. Revise sludge piping.

6.5 LABORATORY CONTROL

The need for examination of sewage as it enters and leaves sedimentation tanks varies from plant to plant. In deciding which tests are essential, the operator must take into account the treatment units which follow, such as trickling filters, aeration tanks, intermittent sand filters, and sludge digestion and dewatering facilities.

Where the effluent is discharged to receiving waters without further treatment, the operator must select tests which will measure the impact on the stream.

6.51 Solids Determinations

6.511 Suspended and Settleable Solids:—Tests for settleable and suspended solids are used to measure the effectiveness of sedimentation. These tests should be made daily on composited samples at large plants and at least twice weekly at smaller plants where laboratory work is conducted.

Samples should be taken of the influent and effluent of the tank at points where the sewage is well mixed. Test results indicate the clarity of the effluent and the loading of suspended matter on the receiving waters or the biological treatment units which may follow. They also indicate the quantity of sludge to be treated and disposed of.

6.512 Total and Volatile Solids in Sewage:—The results of these tests when coupled with suspended solids values give valuable information on the type, concentration, and quantity of solids. Garbage from household grinders, industrial wastes, and infiltration of surface and ground waters may be detected and in some cases estimated from the results of these tests.

Total volatile matter removed in the tanks gives a measure of the loading on sludge-handling facilities and can be used to cross check determinations of volatile matter in the sludge removed.

6.513 Total and Volatile Solids in Sludge:—These tests are preferably performed daily on composite samples of sludge removed from the tanks. The results measure the solids loading on digesters and sludge-dewatering units. Exact knowledge on the per cent moisture in the sludge provides the operator with a way to check his visual observations on the extent of compaction, the need for increasing or decreasing rates and quantities of sludge removed, and the amount of excess water transferred to the digestion and dewatering units.

6.52 BOD

The five-day biochemical oxygen demand is the best single measurement of sewage strength. Tests on composite samples of influent and effluent should be made daily and never less frequently than twice weekly, even in small plants. Tests on the tank effluent provide an excellent measure of the strength and quantity of the organic loading imposed on subsequent biological processes or on the receiving waters.

6.53 pH

The pH of raw sewage should be measured periodically on grab samples to establish the normal range. If the pH is consistently low, it may indicate either that decomposition of the sewage has begun in the sewer system or that partially decomposed or acid waste is being discharged regularly, probably from some industrial process.

The test should be made whenever an unusual waste is seen or suspected.

6.54 Residual Chlorine

When the tank effluent is chlorinated before discharge to receiving waters, the chlorine concentration should be determined with sufficient frequency to assure that the required residual is maintained at all times. Usually one sample per hour will suffice.

When pre- and post-chlorination is practiced, residual tests should be made on samples which reflect the results of each stage of disinfection.

6.55 Coliform Organisms

When the plant effluent is discharged to waters that may be used as a source of drinking water or for swimming, tests for coliform organisms should be made, particularly at the larger plants where facilities and operating staff permit. Daily samples of the effluent and from critical points in the receiving waters should be collected and examined.

6.56 Jar Test for Chemical Coagulation

Where chemical precipitation is used, optimum chemical dosages for effective coagulation are commonly determined by means of the jar test. Since the changing characteristics of the sewage create corresponding changes in chemical dosages, the frequency of testing must fit local conditions.

6.57 Dissolved Oxygen

Dissolved oxygen should be determined to assist in the regulation of air supplied to preaeration facilities that precede sedimentation. Tests on the effluent from the preaeration tanks should be made at least daily, more frequently if changes in sewage strength are to be dealt with.

6.58 Grease

Tests for grease should be made on raw sewage and the effluent from grease removal units. They should be made with sufficient frequency to establish a reliable basis for regulation and adjustment of the treatment units.

6.59 Special Tests

Special tests should be made as required to determine the presence of materials which may create operating difficulties. Such materials include various types of oils and greases, cyanide, copper, zinc, chromium, lead, and other heavy metals. The effect of these materials on plant processes is discussed in other chapters.

Reference

1. Van Kleeck, L. W., "Removing Raw Sludge from Primary Settling Tanks," *Waste Eng.*, **27**, 5, 220 (1956).

TIPS AND QUIPS

Coin Collector

An Associated Press story originating in Baltimore, Md., tells of a city policeman who could hardly believe his eyes as he watched a manhole cover move upward and a self-styled coin collector emerge.

The subterranean prospector admits to finding, over a period of time, nearly $800 with equipment consisting of two flashlights and big pockets. For good pickin', presumably, there is nothing better than a heavy shower, and there is strong evidence that all rain storms in Baltimore can be classified as five-dollar storms, ten-dollar storms, twenty-dollar storms, etc. Up to now, a twenty-dollar storm seems to be tops, but there is usually a jewelry bonus of unreported value.

At the moment, the city itself seems to have turned to picking—picking the coin picker's brain, that is, for more details on his statement that some sewers are in bad condition.

Florida's Best Plants

The winners in Florida's annual awards contest for the best operated treatment plants are: Southwest Orange County, North Palm Beach, Westwood Lake, Coral City, Clearwater, Fernandina Beach, and St. Petersburg. Each award represents a separate plant classification and the presentations were made by Federation Past-President Dave Lee.

Space Saver

J. D. Wittenmyer, vice-president of Huber Utilities, Inc., Dayton, Ohio, writing in *Buckeye Sludge*, the newsletter of the Ohio Conference, describes a solution to a sludge-handling problem.

Sludge in two 25-ft diam final settling tanks was not being scraped to the hopper. The scraper plowed through the sludge all right, but it didn't move the sludge. Sometimes as

Operators of Florida's best-operated plants. L. to r.: D. Joe Raye, Clearwater; Walt Kelly, North Palm Beach; Thomas Wagner and Kinsley Jones, St. Petersburg; and David B. Lee.

much as 3 ft of sludge remained on the tank bottom. Fillets were put in the hopper; they helped, but they didn't solve the problem.

Finally, an attempt was made to move sludge via a direct pump connection to the sludge drawoff line. The drawoff lines are 6 in. in diam and were connected originally to a telescopic valve in the sludge pit. After the pump connection was made, it was found that better results were obtained when only one tank was used. Although the sludge had good settling characteristics, it could not be moved at velocities less than 1 fps.

In a further attempt at solution, the telescopic valves were removed and a 4-in., hydraulically operated, three-way valve was installed between the sludge drawoff lines from the two tanks and the common port connected directly to the sludge pump. The valve is positioned through means of a solenoid valve which is clock operated.

Experience has shown that a 6-min cycle will remove all the sludge from the final tank. Previously, 180,000 gpd of sludge was pumped to the head of the plant. Now only 36,000 gpd is returned. The net saving is 144,000 gpd or capacity enough for 500 additional houses.

Fish-Kill Case Settled *

Litigation between the Virginia State Water Control Board and American Viscose Corporation, Front Royal, Va., has been settled. The Corporation agreed to pay to the Board $30,000 in full and complete settlement of the claim asserted by the Board. The suit was filed July 7, 1959, and involved differences as to responsibility for fish killed in the Shenandoah River in the months of December 1958, and January and February 1959. The Water Control Board claimed that 500,000 fish of different species were killed, and the assessment of $154,777 by the Board against the American Viscose Corporation was based upon the quantity, species, and size of fish killed. In the suit filed by the Board, the Board sought to recover damages in the amount of $154,777. The Corporation denied liability for the fish killed during the period in question.

The American Viscose Corporation has stated that in preparation for trial of the case, it was necessary to employ experts to make protracted studies of a technical nature which would add to the normal costs incident to litigation of this character. For these reasons the Corporation was prompted to settle the case because of the anticipated delay and expense of the litigation.

The proceeds from the settlement will be used by the State Commission of Game and Inland Fisheries to restock the Shenandoah River with game fish as soon as practicable and feasible. The Water Control Board states that this amount will cover the cost of restocking the Shenandoah River with the same number of game fish, i.e., bass, bream, and blue gill, that were killed. The difference in the amount sued for and the amount of the settlement is represented by the estimated value of $74,000 placed upon non-game fish, i.e., catfish, suckers, and carp killed, and the difference between the cost of restocking the river with game fish from the state fish hatcheries and restocking with fish purchased from privately owned hatcheries.

The state has no facilities for nor does it customarily stock or restock any streams with non-game fish. The state fish hatcheries can produce and grow game fish at a cost approximately one-half as great as the price charged by private fish hatcheries. In the opinion of the State authorities, the Shenandoah River will restock itself naturally with non-game fish within a relatively short period of time.

* From the *Conduit*, newsletter of the Virginia Industrial Wastes and Sewage Works Association.

News and Notes

Of Persons and Activities in the Wastewater Field

Ross E. McKinney has resigned from the teaching staff at the Massachusetts Institute of Technology to accept the position of Professor of Civil Engineering at the University of Kansas at Lawrence, Kansas. His principal duties will include the direction of research in the new University of Kansas Environmental Health Center, now under construction.

Max Levine, recently retired as Chief of the Bureau of Laboratories of the Hawaii State Department of Health, was awarded an honorary Doctor of Science degree by the University of Hawaii. He was one of the "Committee of 100" who was credited with the formation of the Federation.

Walter S. Baver, Division Engineer in Charge, Division of Sewers for the City of Newark, New Jersey, died recently.

Frank A. Marston, senior partner in the Boston engineering firm of **Metcalf and Eddy**, received the honorary degree of Doctor of Engineering from Worcester Polytechnic Institute. He is the current President of ASCE.

Francis B. Elder, Engineering Associate on the staff of the American Public Health Association, has received the 1958 Western Hemisphere Award of the USA Section of the Inter-American Association of Sanitary Engineering.

C. S. Boruff, Technical Director of Hiram Walker and Sons, is a newly appointed member of the Federal Water Pollution Control Advisory Board, which is a nine-member group appointed by the President to advise the Surgeon General of the Public Health Service on problems in water pollution.

Lindsey Engineering Co. has moved to 5101 West Fiftieth Street, Minneapolis 24, Minn.

Chester A. Lydecker, Executive Director, and **Sol Seid**, Chief Engineer, Middlesex County Sewerage Authority, have been appointed members of the New Jersey State Advisory Committee on Sewage Disposal.

George F. Fynn, Superintendent of the Buffalo Sewer Authority treatment plant, died recently.

H. Dalton Tomlinson, formerly Principal Sanitary Engineer of the Kentucky Water Pollution Control Commission, is now Professor of Sanitary Engineering at Washington University in St. Louis, Mo.

O. C. Hopkins has been appointed deputy chief of the Public Health Service Division of Water Supply and Pollutuion Control in Washington, D. C. He succeeds **Arve H. Dahl**, who was assigned to the Industrial College of the Armed Forces in Washington.

M. C. Rand has joined the faculty of the Civil Engineering Department of Syracuse University as Associate Professor in the graduate and research programs in Sanitary Engineering. Before assuming his new duties he was Chief Chemist of the Middlesex County Sewerage Authority in Sayreville, New Jersey.

Osmo A. P. Makkonen, of Helsinki, Finland, died recently.

W. F. Welsch, Sanitary Engineer for the Nassau County Department of Public Works, New York, retired this past summer.

P. H. McGauhey, Director of the University of California's Sanitary Engineering Research Laboratory, has been appointed to the Sanitary Engineering Advisory Committee of the California State Department of Public Health.

N. M. deJarnette, formerly with the Georgia Department of Public Health, is now associated with **Robert and Company Associates** in Atlanta, Ga.

Wallace & Tiernan Inc., Belleville, New Jersey, and the R. J. Strasenburgh Company, pharmaceutical manufacturer of Rochester, New York, recently announced an agreement of consolidation and merger.

Frank N. Thomson, Regional Engineer for the New York State Department of Health in Syracuse, died recently.

W. T. Blackman, Sr., Superintendent of the sewage treatment plant in Murfreesboro, Tennessee, died recently. He was the 1958 winner of the Hatfield Award.

C. Franklin Steiner has been appointed district sales engineer for the Pacific Northwest for Process Engineers, Inc., division of the **Eimco Corporation**.

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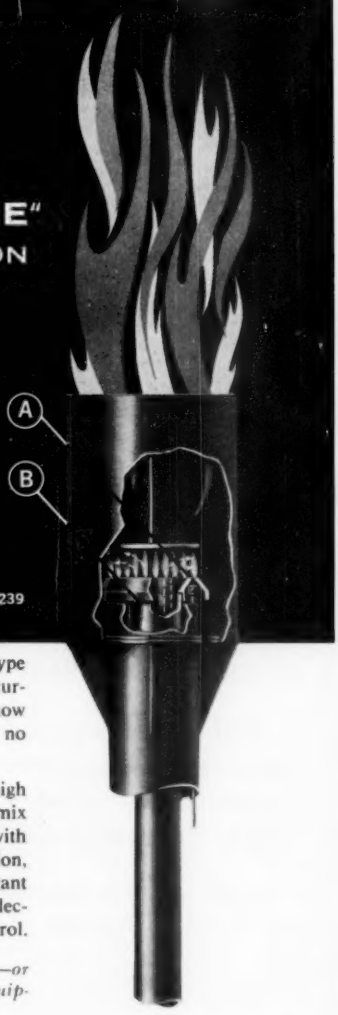

FIG. NO. 239

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Proceedings of Member Associations

ARIZONA SEWAGE & WATER WORKS ASSOCIATION

The Arizona Sewage & Water Works Association, member association of the Water Pollution Control Federation, and the Arizona Section, AWWA, held its annual conference on Apr. 28–30, 1960, at the Pioneer Hotel in Tucson. There were 280 who registered, including 53 ladies.

On Thursday evening at the association dinner, Mr. Byrl Phelps acted as master of ceremonies. There were 16 Arizona Sewage & Water Works Association "Sewage Safety Awards of Merit" conferred on various cities and towns for progress during 1959 in improving and/or initiating safety programs. A signal honor was conferred upon five members of the association by the University of Arizona by presentation to each of the following, a medallion for their individual contributions to the water and/or sewage works fields: John A. Carollo, Phil J. Martin, Guy A. Rhoads, Dario Travaini, and Harold Yost.

There were two speakers at the Friday Luncheon, Gov. Paul Fannin and Ralph E. Fuhrman, Executive Secretary of the Water Pollution Control Federation.

Governor Fannin paid compliment to the Arizona Sewage & Water Works Association and the members individually and collectively for their contribution of knowledge, time, and effort toward assuring an adequate supply of pure water and the safe disposal of sewage and industrial wastes both in the past and for the future.

Officers of the Arizona Sewage & Water Works Association: standing, l. to r., A. D. Cox, Jr., Secretary-Treasurer; R. E. Polenske, President; Stanford I. Roth, 1st Vice-President; seated, l. to r., H. E. Bigglestone, Past President; Milford E. Devine, 2nd Vice-President.

E. O'Neal Dye, Superintendent, Tucson sewage treatment plant, was the recipient of the Hatfield Award. His citation read: "To E. O'Neal Dye, the William D. Hatfield Award for his extraordinarily complete records con-

(*Continued on page 551a*)

WPCF publications

Safety in Wastewater Works

Contains information on hazards, accident statistics, rescue methods, safe practices, and equipment. *56 pages. $0.75 to members; $1.50 to others. Order as Item MOP-1.**

Utilization of Sewage Sludge as Fertilizer

An authoritative evaluation of advantages and limitations of sludge as a soil conditioner. *120 pages. $0.75 to members; $1.25 to others. Order as Item MOP-2.**

Chlorination of Sewage and Industrial Wastes

Reviews history, development, and up-to-date technique of chlorine application. *90 pages. $1.00 to members; $1.25 to others. Order as MOP-4.**

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History and modern practice in design, installation, operation, and maintenance of diffuser systems and appurtenances. *75 pages. $1.00 to members; $1.25 to others. Order as Item MOP-5.**

Units of Expression for Wastes and Waste Treatment

Lists of recommended units for quantities ordinarily encountered in wastes and waste treatment. *8 pages. $0.25 to members; $0.50 to others. Order as Item MOP-6.**

Sewer Maintenance

A guide for maintaining a municipal or industrial drainage system in serviceable condition. *64 pages. $1.00 to members; $1.50 to others. Order as Item MOP-7.**

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Prepared jointly by the Federation and ASCE. Detailed presentation of the subject. *375 pages. $3.50 to members; $7.00 to others. Order as Item MOP-8.**

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WATER POLLUTION CONTROL PRODUCT GUIDE

This list of products and services is offered as an information aid. The list was derived by solicitation from those firms listed. The JOURNAL will make every effort to keep this list current with the help of members and advertisers. All Associate Members and all advertisers for the past year are included. Many of the services and products are advertised in this issue. For an alphabetical listing see Index to Advertisers in back of JOURNAL.

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cerning the Tucson sewage treatment plant."

Art F. Vondrick, Utilities Engineer, Water and Sewage Department, city of Phoenix, was the recipient of the Bedell Award. Art's citation read: "To Arthur F. Vondrick, the Arthur Sidney Bedell Award for his outstanding work within the member association, Arizona Sewage & Water Works Association."

Papers presented during the conference included:

"Space Sanitation," by Miss Franki Laurel Van Der Wal, Member, Research & Development Division, Space Technology Laboratories, Inc., Los Angeles, Calif.

"Operation and Maintenance Costs of Sanitary Sewers," by Art F. Vondrick, Utilities Engineer, Water & Sewage Department, Phoenix, Ariz.

"New Standards—Water and Sewage," by Dr. Karl Hirlinger, Field Chemist, Arizona State Department of Health, Phoenix, Ariz.

"Settling Tank Maintenance at Hyperion," by Glenn W. Thwaytes, Mechanical Superintendent, Hyperion Treatment Plant, Los Angeles, Calif.

"Instrumentation—Its Purpose and Results in the Water and Waste Field," by Richard M. Day, Field Engineer, Fischer & Porter Company, Anaheim, Calif.

"Urban vs. Rural Pollution Affecting Receiving Waters in Imperial Valley," by Harvey F. Ludwig, President, Engineering Sciences, Inc., Arcadia, Calif.

"Sand Removal by Liquid Cyclones," by William E. Budd, Manager, Sanitary Division Development Department, Dorr-Oliver Inc., Stamford, Conn.

Officers elected at the annual business meeting were:

(*Continued on page 553a*)

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NEW ENGLAND SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

Spring Meeting

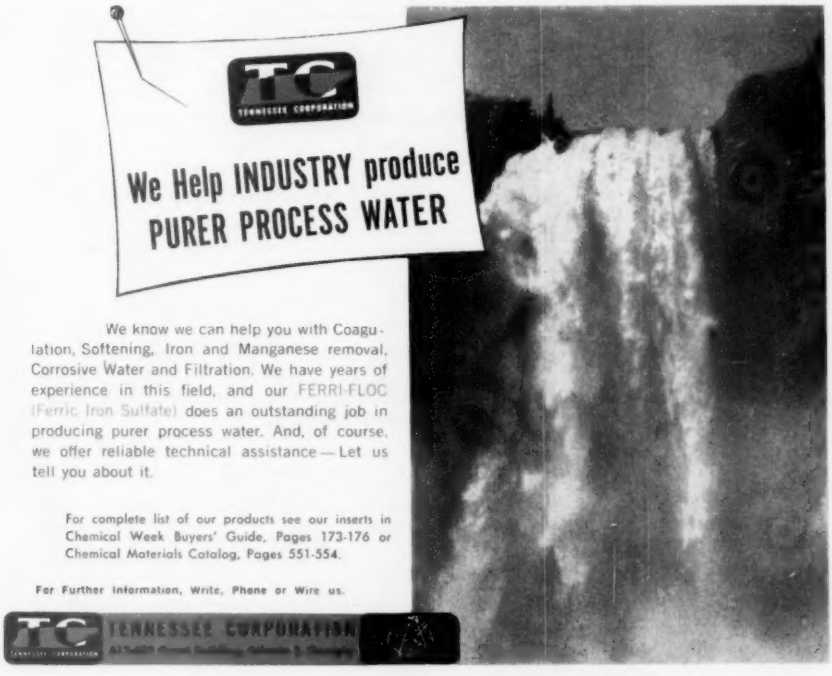
The 1960 spring meeting of the New England Sewage and Industrial Wastes Association was held at the Sheraton-Kimball Hotel, Springfield, Mass., May 25–26, 1960. Total registration included 155 members and guests, and 21 ladies.

The technical program began with a joint session on "Temperature Requirements for Odor Destruction during Incineration" by Clair N. Sawyer and Peter A. Kahn of Metcalf & Eddy. Following this paper, LeRoy W. Van Kleeck of Bowe, Albertson & Associates spoke on the new Longmeadow, Mass., sewage treatment plant.

Later, those attending divided into two groups. The designer's section heard papers on: "Private Sewage Disposal Needs" by J. Henry L. Giles, consulting engineer, and I. Laird Newell, President, Henry Souther Engineering Co.; "Volatile Acids by Direct Titration" by Orris E. Albertson, Dorr-Oliver Company, and Miss Rosemarie DiLallo, Chemist, New Rochelle, N. Y., sewage treatment plant; and "Analytic Toxicology of Industrial Effluents" by Mark M. Luckens, Connecticut State Department of Health.

The operators section heard papers on: "Sludge Line and Heat Exchanger Cleaning and Maintenance" by Robert W. Giddens, Assistant Superintendent,

(*Continued on page 555a*)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Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
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Rockwell Mfg. Co.
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National Clay Pipe Mfrs., Inc.
National Sewer Pipe Ltd. (Canada)
Perry-Austen Mfg. Co.
Robinson Clay Product Co.
Stebbins Engr. & Mfg. Co.
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Nut Island sewage treatment plant, Boston, Mass.; "Electric Motor and Control Maintenance" by W. H. Brackett, W. H. Brackett Company; and "Installation, Maintenance and Operation of Sluice Gates" by Roy Soderberg, Superintendent of Field Services, Rodney Hunt Machine Company.

The spring meeting luncheon was attended by 141 members and guests. Following it, inspection trips were made to the Longmeadow sewage treatment plant and the Springfield main sewage treatment plant at Bondi Island, Mass.

EARL R. HOWARD
Secretary-Treasurer

KANSAS SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 15th Annual Meeting of the Kansas Sewage and Industrial Wastes Association was held in conjunction with the Kansas Section of the American Water Works Association at the Broadview Hotel, Emporia, Kans., on April 20–22, 1960. Registration for the meeting totaled 213, including 25 ladies.

Sewerage practice held the limelight at the technical sessions. Among the papers were these:

"Relationship of the City, the Consulting Engineer, and the Contractor on Utility Construction Projects," panel discussion.

"Problems Confronting the Consulting Engineer in Designing New Improvements for a Municipality," by O. O. Ediger, Ediger Engineering Co., Wichita, Kans.

"Four Parties and a Maintenance Clause," by J. H. Holloway, Salina Supply Co., Salina, Kans.

(*Continued on page 557a*)

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"A Study of Storm Water in Sanitary Sewers," by L. W. Weller, Black & Veatch, Kansas City, Mo., and Myron Nelson, Chief Engineer, Mission Township Main Sewer District, Mission, Kans.

"Experiences at Our New Sewage Treatment Plant," by Leonard Beyer, Superintendent, Sewage Treatment, Hutchinson, Kans.

"Sewer Construction Practices," by F. G. Writer, Clay Products Association, Topeka, Kans.

"Sludge Handling and Disposal," panel discussion.

"Digester Gas Systems," by Melville Gray, Kansas Board of Health, Salina, Kans.

"Maintenance of Electrical Equipment," by Leland Talbot, Superintendent of Maintenance, Sewage Treatment Plant, Wichita, Kans.

"Status of Retirement Program for Public Employees," by F. D. Diehl, Superintendent of Utilities, McPherson, Kans.

A field trip to the Emporia sewage treatment plant was conducted by Lee Stolfus, city engineer. Federation activities were discussed by Ray E. Lawrence, WPCF Vice-President.

At the business meeting the Association's name was changed to the Kansas Water Pollution Control Association. New officers for 1960–61 are:

Chairman: Harry Pfautz, Hesston.
1st Vice-Chairman: John Glidewell, Topeka.
2nd Vice-Chairman: N. J. Burris, Topeka.
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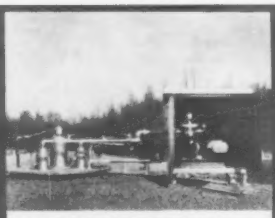

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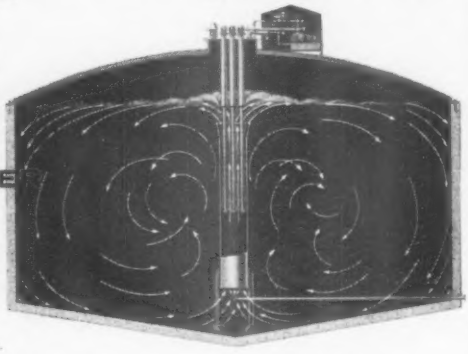

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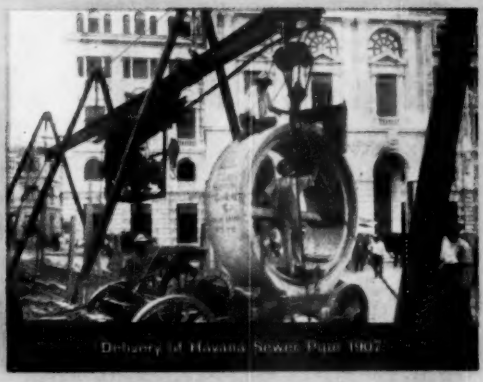

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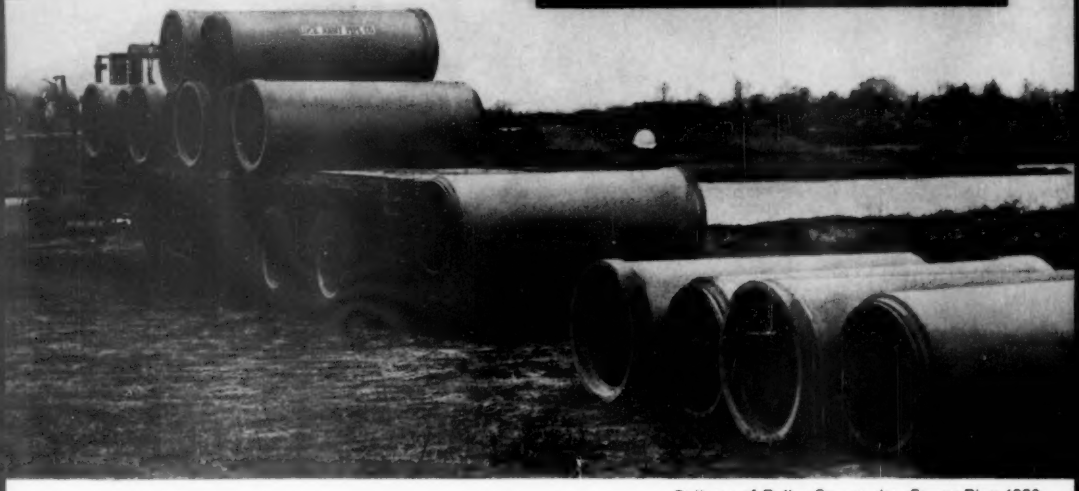

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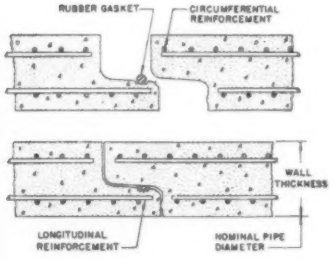

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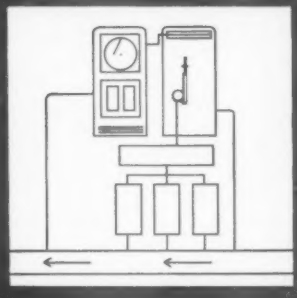

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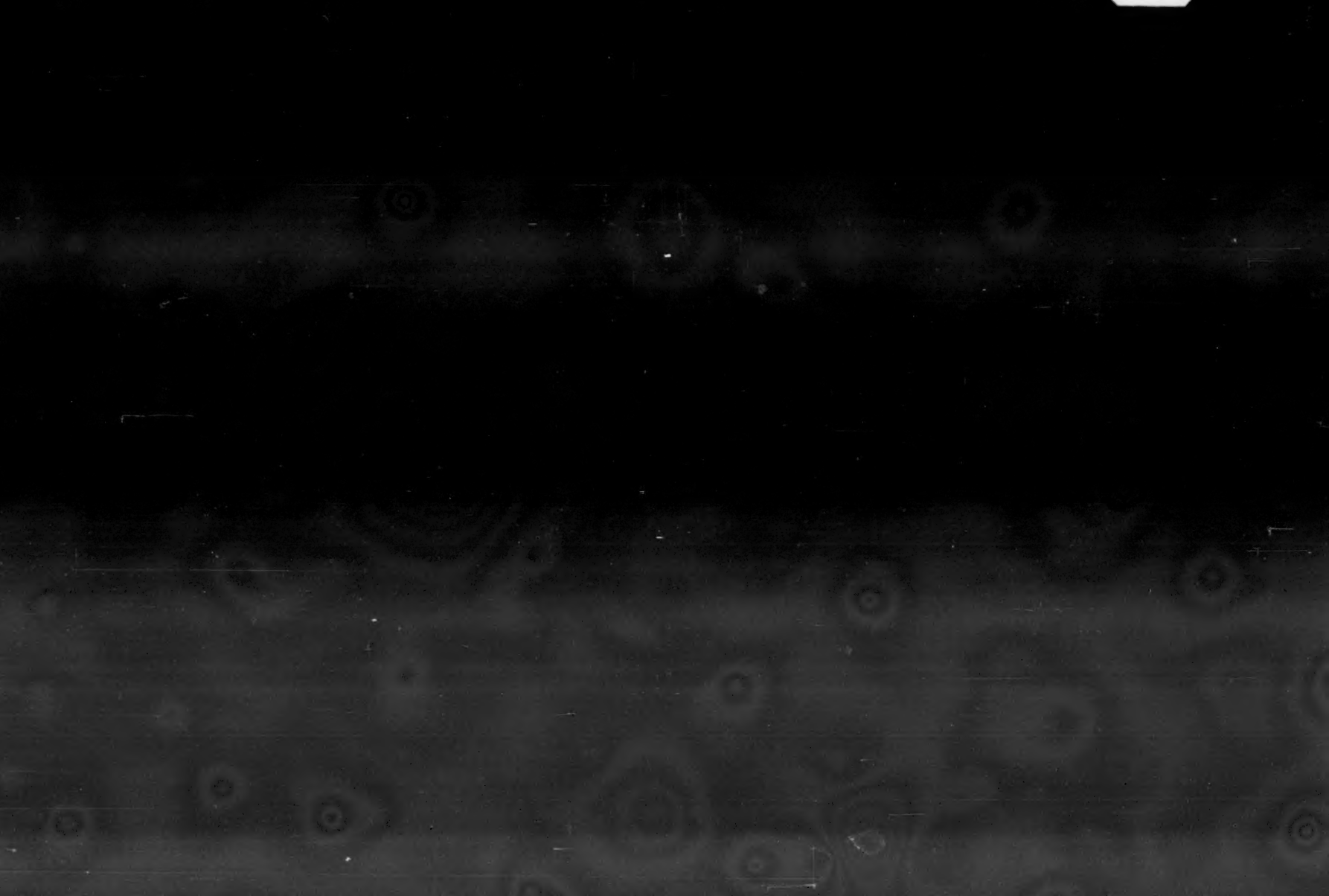
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